AN EXPERIMENTAL INVESTIGATION

OF

THE 'PREFLEX' METHOD

OF

PRESTRESSING CONCRETE

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AN EXPERIMENTAL INVESTIGATION

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Frank Allen Mink, Jr.

Pottstown, Penna.

May 3, 1955

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AN EXPERIMENTAL INVESTIGATION OF THE "PREFLEX" METHOD OF PRESTRESSING CONCRETE

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ABSTRACT

The purpose of this investigation was twofold. The first purpose was to determine whether the elastic theory, as it applies to beams of two materials, can be used to safely and accurately predict the stresses and deflections which will occur in a beam which is prestressed by the preflex method. The second purpose was to determine what, if any, increase in moment carrying capacity is produced in a beam by preflexing it, when compared with the capacity of a plain steel beam encased in concrete.

In the course of the investigation, two beams were prestressed by preflexing. Plain steel beams, simply supported, were loaded at their third points, and stressed to 70 percent of their yield strength. While so loaded, high early strength concrete was placed and allowed to cure around the tension flange. After curing the concrete for seven days, the loads were removed. Elastic recovery of the steel beams prestressed the concrete to approximately 2800 p.s.i.

After the loads were removed, concrete was placed around the compression flange of one of the beams. The other beam had no concrete placed around its compression flange.

A third beam, not prestressed was constructed as a control

beam. It was constructed of the same materials and to the same dimensions as the preflexed beam with top flange concrete.

Each beam was subjected to a load test which measured its strength and deflection properties. Loads were applied at the third points of each beam, and increased gradually until the beam failed. Strain and deflection of each beam were measured for each increment of load.

Strain measurements indicated that the elastic theory can be safely used to predict stresses in preflexed beams during the preflexing operation and during the service life of the beam. Measurements taken to determine the amount and type of prestress loss were inconclusive, and no recommendations concerning this subject can be made at this time.

The load tests indicated that preflexing greatly increased resistance to cracking of concrete in the tension flanges of the beams. The load which was necessary to cause cracking was more than doubled by preflexing. The tests also indicated that beam strains and beam deflections were substantially reduced by preflexing. Preflexing had no effect upon the ultimate strength of the composite beams.

As a result of the investigations, a tentative procedure for the design of preflexed beams was developed. Stresses which will occur in the various phases in the construction and service life of a preflexed beam are computed by the conventional flexural formula, and the transformed area method. The values of the sectional properties used in this formula are computed for an equivalent steel section for each phase in the life of a preflexed beam. The stresses caused in each phase are added to obtain the critical stress condition which governs the dimensions of the preflexed beam.

Future research should be directed toward determination of the amount of prestress loss which will occur in a preflexed beam, and toward the effects of long time loads.

Approved ______________________

Harry C. Saxe
Chairman

Date Approved ______________________

CHAPTER I

INTRODUCTION

Statement of the problem.--The purpose of this investigation was (1) to determine the accuracy with which the elastic theory may be used to predict the stresses occurring in a beam prestressed by the preflex method, and (2) to compare the resistance to deflection and cracking of a preflexed beam with that of a similar non-preflexed beam.

History of the problem.--Often, in the design of a structure, it is desirable to encase a steel beam completely in concrete. This may be necessary because of space limitations, architectural considerations, fire-proofing, corrosion conditions, or a tendency of the steel to buckle. When the steel is covered by concrete in the conventional manner, the concrete on the tension flange of the composite beam cannot be counted on for any moment resistance because of the extremely low tensile strength of concrete.

As a result of this inherent weakness of the concrete, crack formation in the tensile region of ordinary composite beams is unavoidable. These cracks are detrimental from the standpoint of structural efficiency, in that they permit corrosive materials to come in contact with the steel. Moreover, such cracks when visible are unsightly, and detract from the esthetic qualities of such beams.

The development of prestressed concrete by Magnel, Freyssinet, Abeles, and others (1), (2), (3) has solved the problem of tension cracks in normal reinforced concrete beams by subjecting the tension area of a beam to a previously applied compressive stress. In general, this method requires that individual wires or cables be tensioned by jacking either before or after the concrete is placed and cured. This method has even been used in conjunction with rolled steel beams encased in concrete, but the steel beams were not used to apply the prestress to the concrete. This method of prestressing concrete requires complex jacks, bearing plates, cable anchorages, and forms, and is very often too expensive to compete with normal reinforced concrete in the United States.

In September 1951, two Belgian engineers, L. Baes and A. Lipski (4) developed a unique method of prestressing the concrete covering around a steel beam. The method consisted of preloading a plain steel beam with its eventual working load, reproducing the working load moment diagram as nearly as possible. Then while the beam was loaded, concrete was placed around the tension flange and allowed to cure. After the concrete had fully cured, the beam was unloaded. The elastic straightening of the steel beam applied compressive stress to the concrete by means of bond between the steel and the concrete. After the beam was unloaded, concrete was placed around the other flange, so that the steel was completely encased in concrete.

The elastic theory as it applies to beams of two materials of different elastic properties was used to compute the stresses which controlled the design of the preflexed beams (5). The equivalent cross section of the beam was determined, and its sectional properties computed. Stresses which were produced in the preflexed beams were computed by the conventional flexural formula. This expression was used to compute all the critical stresses which would occur during the prestressing of a preflexed beam, and during its service life. The accuracy of the design computations was not checked experimentally by Baes and Lipski during tests on the actual beams.

Purpose of the research.--The first purpose of the research described herein was to determine the accuracy with which the stresses and deflections which occur in a preflexed beam can be predicted by the elastic theory. The second purpose was to discover what increase in rigidity and moment-carrying capacity is obtained by preflexing a steel and concrete beam.

The preflexed beams constructed and tested by Baes and Lipski were analyzed by the elastic theory, without the applicability of the elastic theory being determined. At no time during the tests described (4) were actual strain measurements compared with stresses predicted by the elastic theory. Neither did the authors refer to or describe any previous tests which had established the applicability of the elastic theory to the calculation of stresses in beams prestressed by the preflex method. The first purpose of this investigation, then, was to determine whether the elastic theory can be used to accurately compute stresses which would occur during the various phases of prestressing a steel and concrete beam by the preflex method.

In order to test the reliability of the elastic theory, stresses and deflections for a beam of known dimensions and materials were computed using this theory. These stresses and deflections were computed for many important phases of the construction and working life of preflexed beams. The computed stresses and deflections were compared with actual strains and deflections measured when each of two preflexed beams were constructed and tested by the author.

The preflexed beams tested by Baes and Lipski (4) were claimed to have greatly increased resistance to cracking and deflection, when compared with the resistance to cracking and deflection of similar, non-preflexed beams. Since those tests were made using European design methods, materials, and construction practices, it was felt that similar tests using American practices would increase American understanding of the preflex method. The second purpose of this investigation, then, was to compare the cracking and deflection characteristics of a preflexed beam, with the cracking and deflection characteristics of a non-preflexed beam with the same dimensions and materials.

Survey of the literature.--In September, 1951, two Belgian engineers, Louis Baes and A. Lipski, published their first two papers dealing with the prestressing of concrete by the preflexing method. The first paper (4) reported the results of comparative tests run on a preflexed and on a non-preflexed steel-and-concrete beam. The second paper (6) presented methods to be used in the analysis and design of preflexed beams. The methods of analysis were based on the elastic theory of beams of two materials having different elastic properties. The design methods were semi-empirical in nature, especially with respect to bond, shear, and prestress loss.

Two subsequent papers (7), (8), published in 1953, described construction methods used in Belgium in the production of preflexed beams. The papers also described structures in which beams prestressed by preflexing have actually been used.

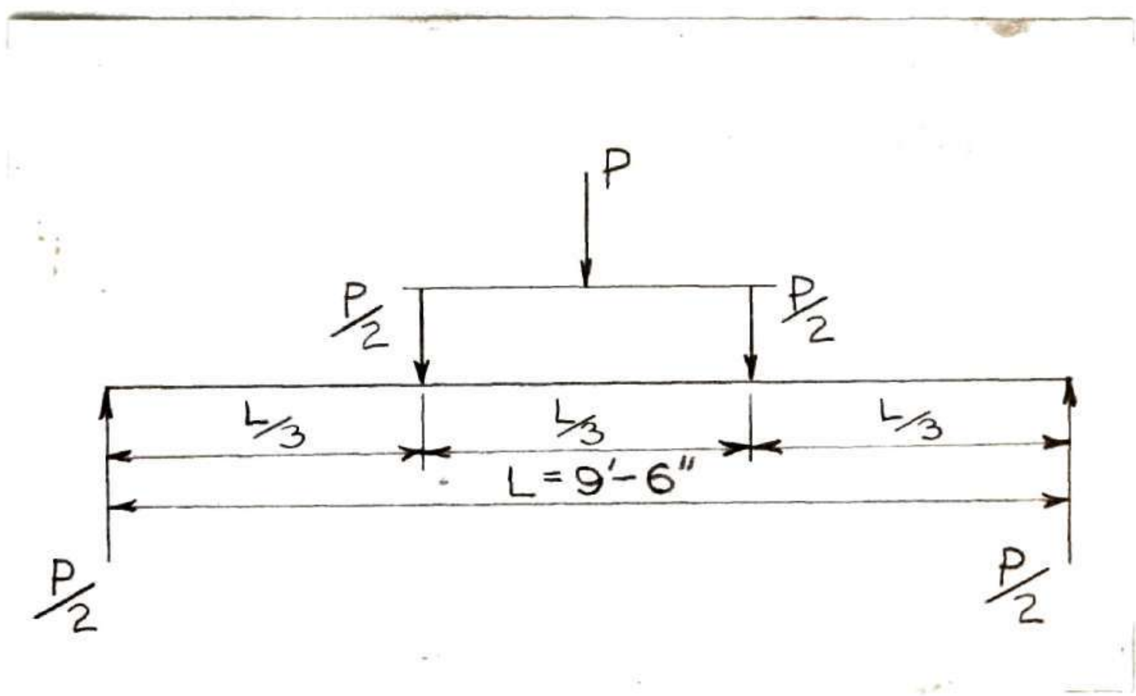
There have been no papers dealing with the preflex method published in the United States to date. The only publications in the English language are fragmentary British reviews (9) of the Belgian publications (4), (5) previously cited.

CHAPTER II

THEORETICAL CONSIDERATIONS

The methods used to compute the state of stress which existed during each stage in the construction and testing of the preflexed beams were semi-empirical. The compressive strength and the modulus of elasticity of the concrete used were determined from the results of standard tests. The tests were made on representative test cylinders cast from concrete which was used in the construction of the preflexed beams. Every effort was made to use concrete with identical properties in the construction of the two preflexed beams, and the non-preflexed control beam. Due to the variation of materials, some difference in ultimate strength of concrete used in the three beams occurred. These minor differences were averaged before being used to compute the sectional properties of the preflexed beams.

Preflexing stresses in the plain steel beam.-- The steel used in construction of the preflexed beams had a yield strength of 55,960 p.s.i., as shown by tests made by Bethlehem Steel Co. (See Table 3) in Appendix A. The steel beam was an 8 WF 17, with an overall length of 10'- 0''. The beam was loaded at the third points of a span of 9'- 6''.



Position of Preflexing Loads

Figure 1

Total load, P, used in preloading the steel beam was 28,900 pounds. The section modulus, S, of an 8 WF 17 is 14.1 in^3. Combining the common flexure formula:

$$f = \frac{M}{S} \tag{1}$$

where: f = unit stress (p.s.i.)

M = applied bending moment (lb-inches)

S = section modulus (in^3)

and the expression for bending moment in the center of a simply-supported beam loaded at the third points:

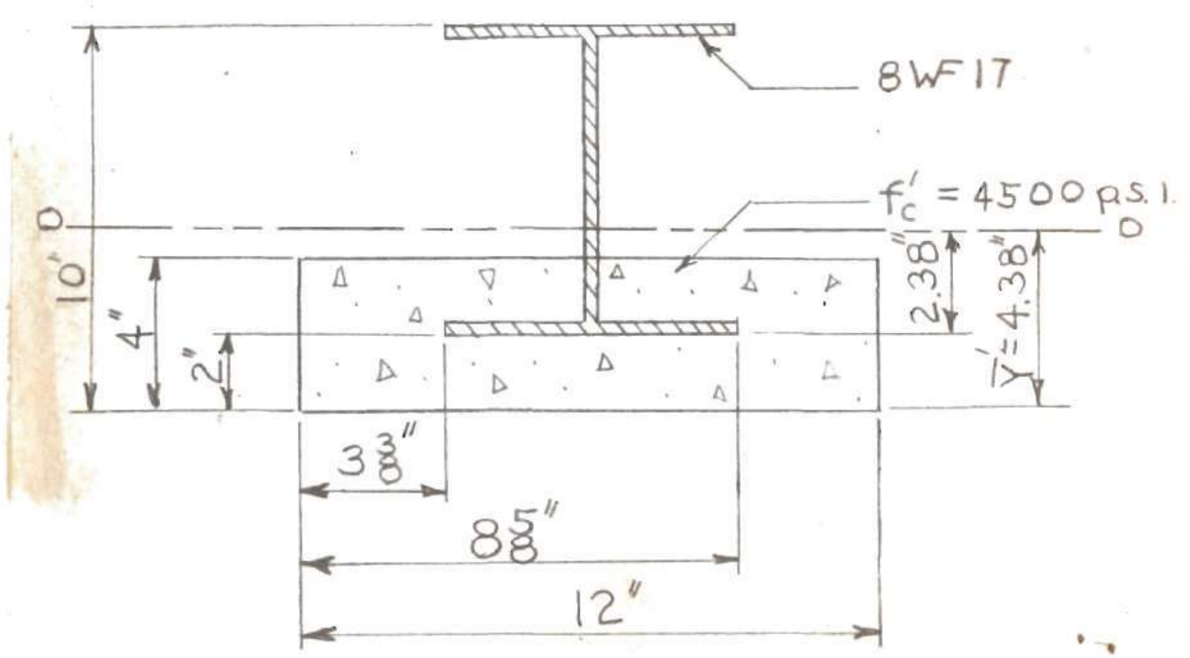
$$M = \frac{PL}{6} \tag{2}$$

we obtain the relationship between maximum stress and load for the pre-loaded steel beam:

$$f = \frac{PL}{6S} \tag{3}$$

Maximum steel stress caused by the preflexing load was computed by this expression as 39,000 p.s.i., or approximately 0.7 of the yield strength.

<u>Stresses in the steel and concrete beam after unloading</u>.--While the steel was stressed to a maximum of 39,000 p.s.i., concrete having an ultimate cyclinder strength at seven days of 4500 p.s.i., and a modulus of elasticity, E_c, of 2.76×10^6 p.s.i., according to tests, was placed around the lower steel flange, forming a composite beam with dimensions as shown in Figure 2.



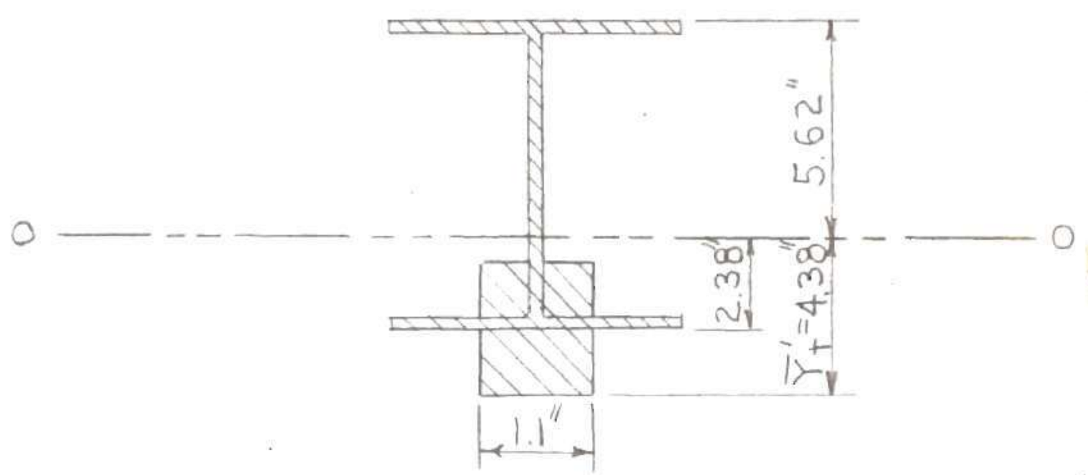
Beam Section at Removal of Preflexing Load

Figure 2

The modulus of elasticity of the steel, E_s, was assumed to be 30×10^6 p.s.i. The modular ratio, n, between the steel and the concrete composing the beam, as computed by the formula:

$$n = \frac{E_s}{E_c} \tag{4}$$

was equal to 10.9. This modular ratio was used to transform the lower flange concrete area into an equivalent steel area, giving an equivalent steel section of the dimensions shown in Figure 3.

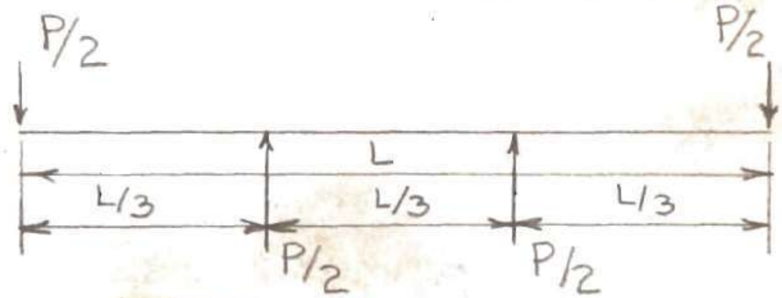


Transformed Section

Figure 3

The location of the centroidal axis, and the moment of inertia of the equivalent steel section were then computed. The moment of inertia of the transformed section, I_t, was computed as 99.9 in^4. The distance from the bottom of the transformed section to the centroidal axis, Y_t, was computed as 4.38", as shown in Figure 3.

Using the sectional properties obtained above, the changes in steel and concrete stresses caused by unloading the composite beam were computed. The effect of removing the preflexing load was represented mathematically by applying a load equal to the preflexing load, but oppositely directed, as shown in Figure 4.



Removal of Preflexing Load

Figure 4

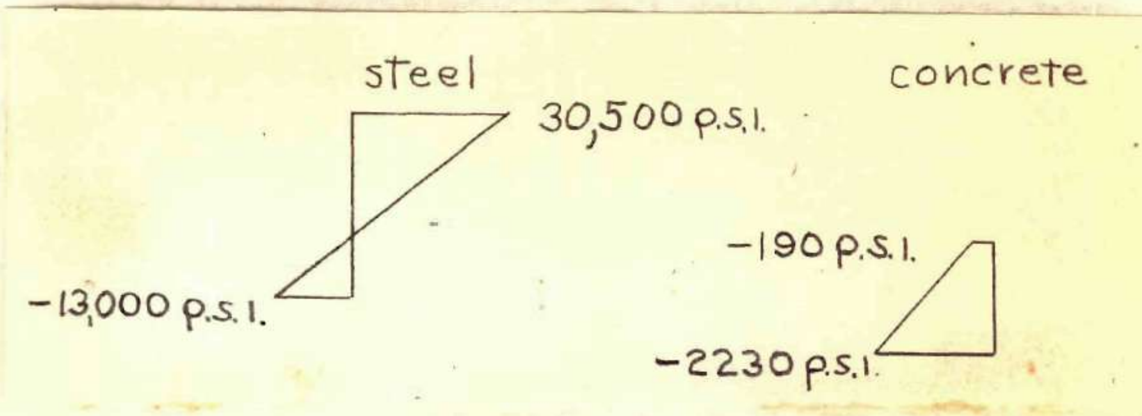
Changes in steel stresses, f_s, caused by the removal of the preflexing load, were computed by combining the flexure formula Equation (1), and the formula for bending moment at the center of a beam loaded at the third points, Equation (2), resulting in the relationship

$$f_s = \frac{PLC}{6I} \tag{5}$$

Changes in the concrete stresses, f_c, caused by removal of the preflexing load, were computed by formula (5), modified by the modular ratio of steel and concrete, n, or:

$$f_c = \frac{PLc}{6In} \tag{6}$$

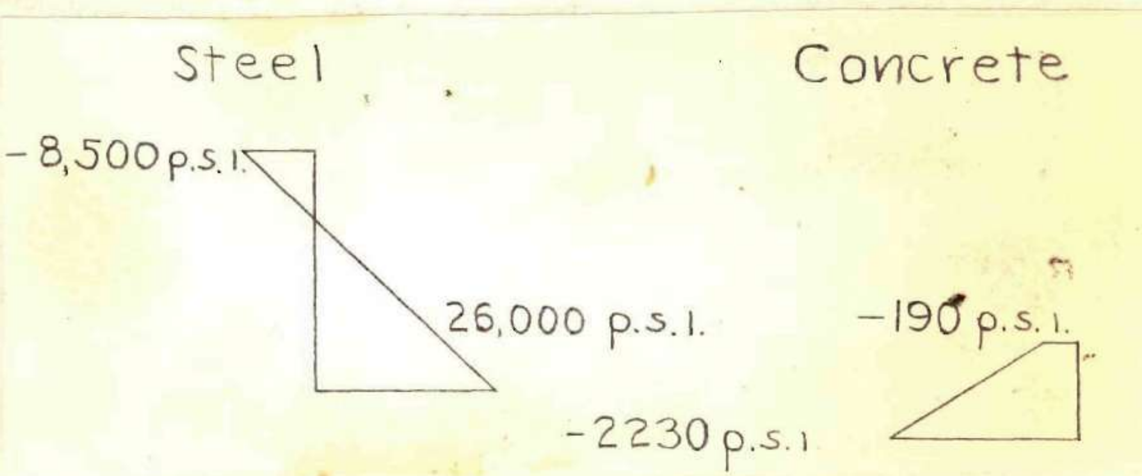
Changes in stresses caused by removal of the preflexing load were computed to have the following values.



Stress Changes Caused by Removal of Preflexing Loads

Figure 5

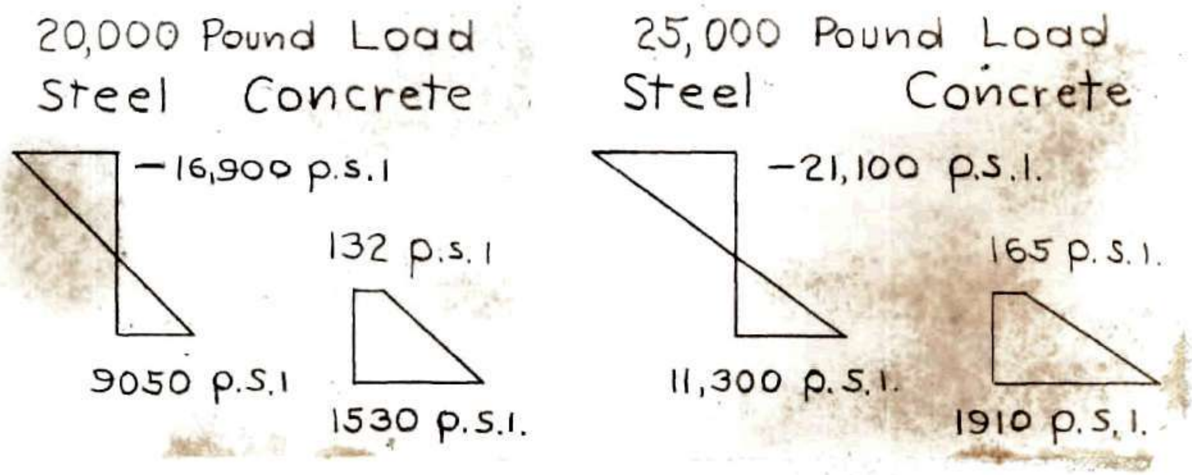
These changes in stresses, added to the original steel stress caused by the preflexing load, caused theoretical stresses after the preflexed beams were unloaded, to be as shown in Figure 6.



Stresses at Removal of Preflexing Load

Figure 6

The concrete covering around the upper steel flange and the web was omitted from one of the preflexed test beams. When this beam was subjected to a load test, its sectional properties were the same as they were when the preflexing load was removed. The theoretical changes in stresses due to two of the loads applied during the load test on this beam were computed and are summarized below.



Changes in Stresses Caused by Test Loads on Beam #3

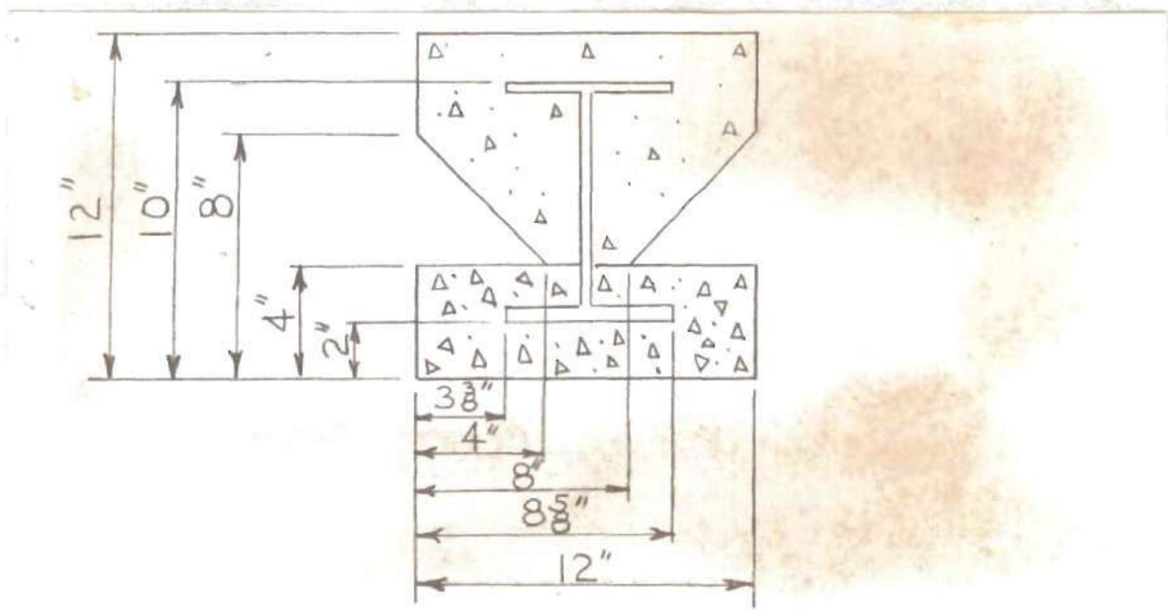
Figure 7

No attempt was made to compute the theoretical stresses in the preflexed beam at the beginning of the load test, because there was no theoretical method of predicting the amount of prestress loss which would occur. Appreciable prestress loss due to shrinkage and plastic flow in the concrete was expected to occur between the time of removal of the preflexing load and the execution of the load test. Prestress loss was observed by means of strain readings taken after the removal of the preflexing load. At the time the beams were analyzed, it was impossible to predict the theoretical magnitude of the prestress loss. In order to provide a consistent comparison with the results of the load tests, only theoretical *changes* in stress caused by changes in applied load were computed.

Stresses in the steel and concrete beam after placing of top flange concrete.--After removing the preflexing load of one of the beams, concrete with an average seven day ultimate cylinder strength of 5000 p.s.i. was placed around the upper steel flange of the beam. This concrete had an experimentally obtained modulus of elasticity, E_c', of 2.28×10^6 p.s.i. The modular ratio, n', between this concrete and the steel was computed from the expression:

$$n' = \frac{Es}{E_c'} \tag{7}$$

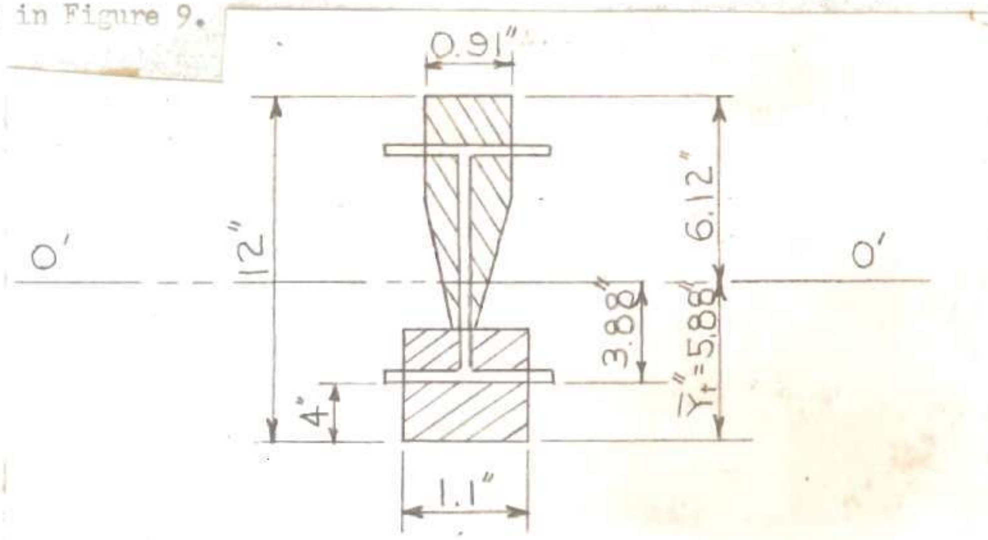
as 13.1. The actual cross section of the beam, with top concrete in place, had the dimensions as shown in Figure 8.



Final Beam Section

Figure 8

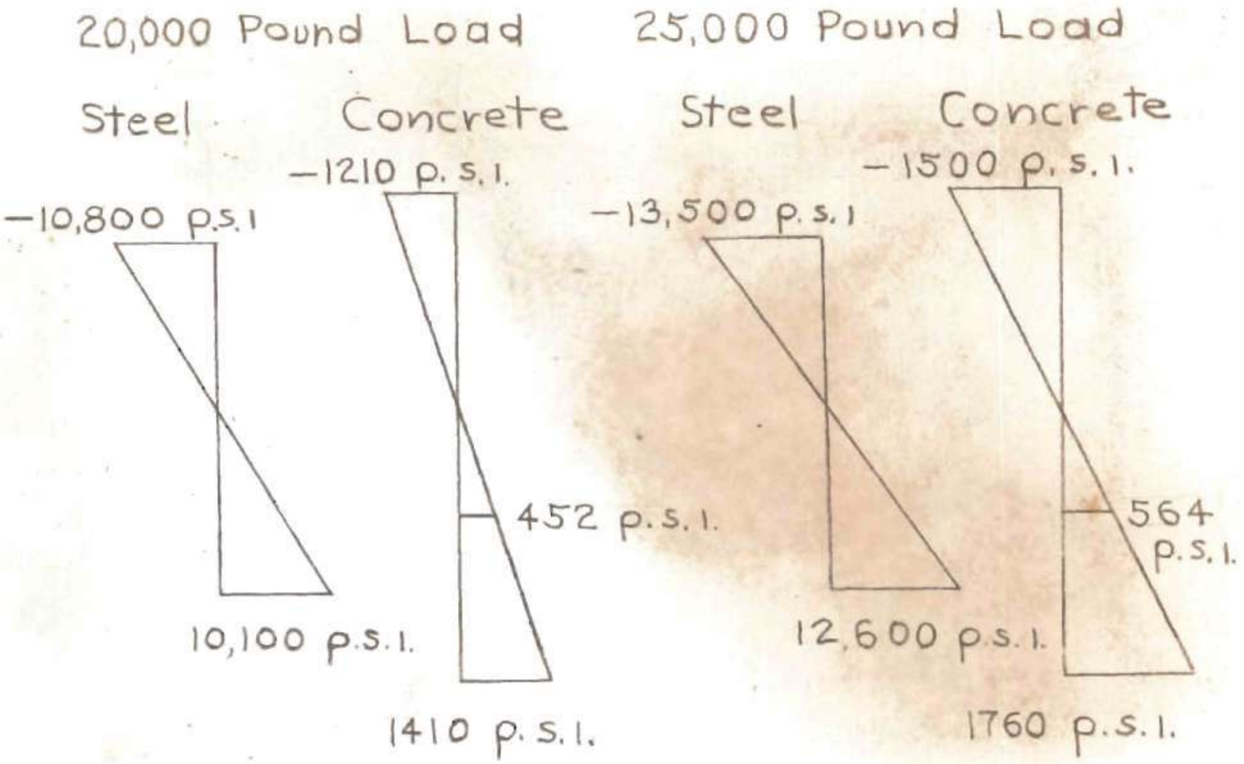
The area of the top flange concrete, 80 $in.^2$, when divided by the modular ratio, n', produced an equivalent steel section with the dimensions shown in Figure 9.



Final Transformed Section

Figure 9

The distance from the bottom of the lower concrete flange to the centroidal axis, $\bar{Y}_t'$, was computed as 5.88 in. The moment of inertia of the entire transformed section, I_t', equaled 145.6 $in.^2$ Theoretical changes in stress due to two changes in load during the load test were computed by equations (5) and (6). These computed stresses are significant only below the loads at which the lower flange concrete cracked, that is only at loads where the entire cross section of the composite beam was effective in resisting bending moment. The computed changes in stress due to changes in load are shown in Figure 10. Again, computations were made only of changes in stresses caused by changes in load, because of the inability to predetermine the amount of prestress loss, and consequently, the amount of prestress remaining when the load test was begun.



Theoretical Changes in Stresses

Figure 10

The control beam had the same sectional properties, initially, as did the second preflexed beam. The only difference was that it was not prestressed. This meant that the tension flange concrete would crack under small load, and that after it cracked, the moment of inertia of the section would be reduced. Since the moment of inertia of the non-preflexed control beam would be constantly decreased as the neutral axis moved upward under increasing loads, no attempt was made to compute theoretical changes in stress in the control beam.

Ultimate loads.--No theoretical investigation was made of the actions of the preflexed beams or the control beam under loads approaching the ultimate. By observation, however, it may be noted that prestressing a beam should have no effect on its ultimate load. This is true because once the precompression in the tension flange of a beam becomes zero, all effects of prestressing are lost as the load increases. Once a prestressed section cracks, its moment resisting capacity is the same as that of an identical non-prestressed section. Therefore, the ultimate loads of the control beam, and of the preflexed beam with top flange concrete, should be of the same magnitude.

It is also true that the ultimate load of the preflexed beam with lower flange concrete only, should be of the same magnitude as that of a similar plain steel beam. This is true because once the prestressed lower flange concrete cracks, it is of no further moment resisting value. Only the steel is then available to resist any increase in bending moment. For this reason, the ultimate loads of the preflexed beam without upper concrete and a plain steel beam should be approximately equal.

CHAPTER III

INSTRUMENTATION AND EQUIPMENT

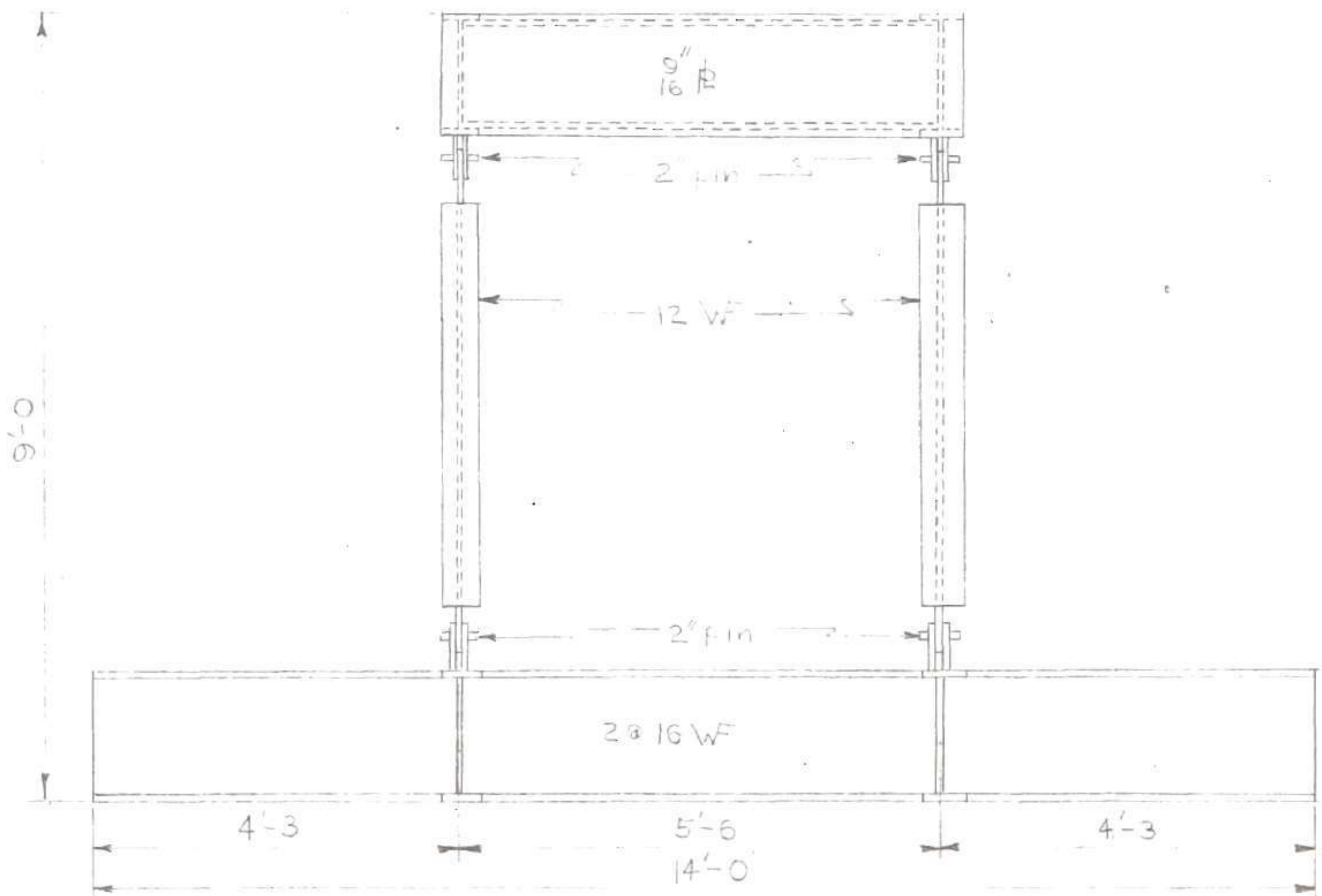
Loading Equipment

The equipment used to apply the preflexing loads and the test loads to the beams tested in this investigation consisted of a loading frame, hydraulic jack, four screw jacks, and a proving ring.

Loading frame.--The frame used to apply the load to the beams in all tests was rectangular in shape, and constructed of structural steel sections and plates as shown in Figures 11 and 12. The frame was of welded construction, but major components were pinned together to allow it to be disassembled and moved. A transverse load could be applied to members with a maximum span length of 14 feet. Loading capacity of the frame was 200,000 pounds.

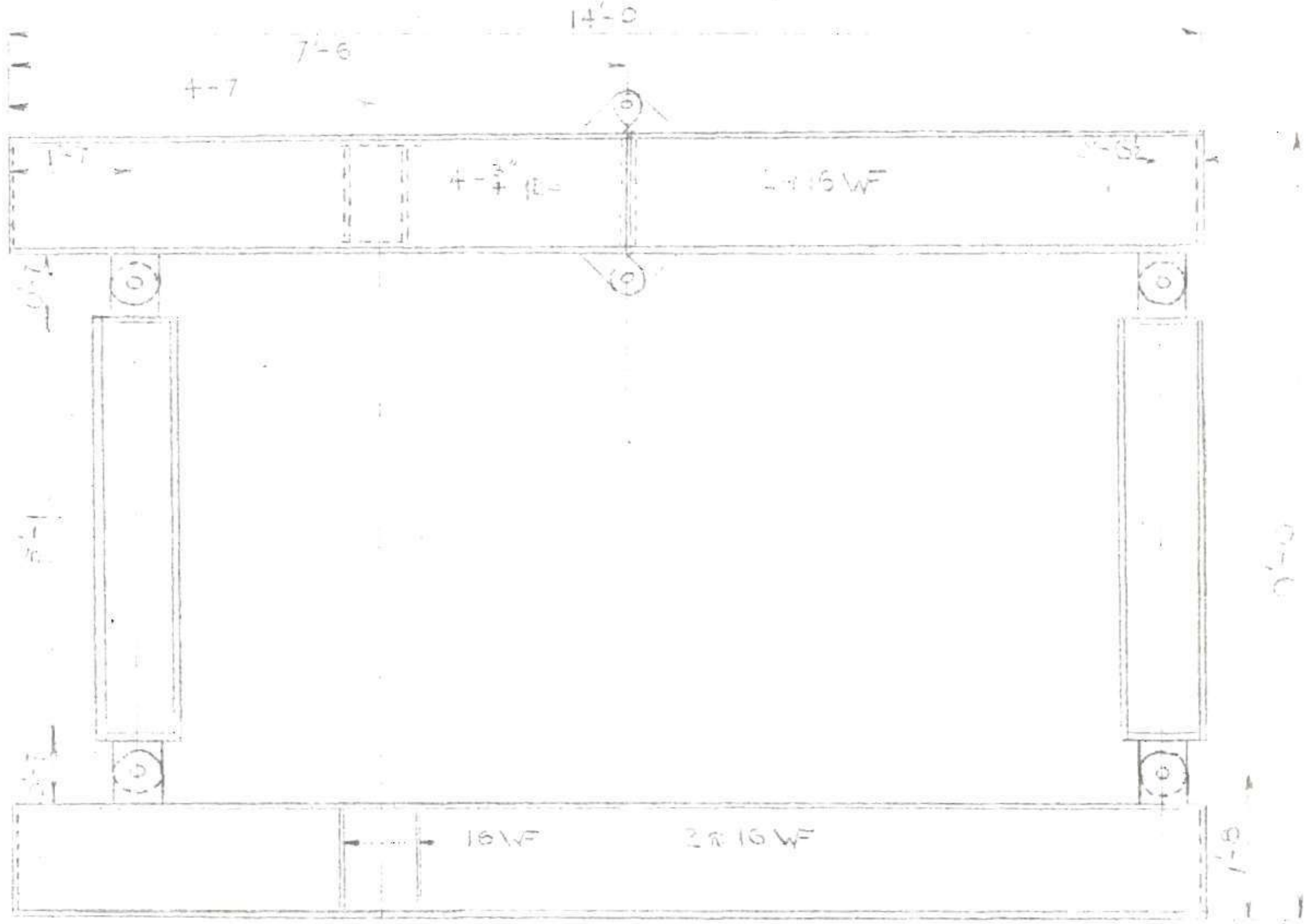
Hydraulic jack.--Attached to the overhead beam in the center of the loading frame was a sixty ton capacity hydraulic jack. This jack was inverted and had a proving ring threaded onto its lower end. The hydraulic jack was used to apply short-time loads during the final testing of all beams, but due to the difficulty of maintaining constant loads with hydraulic loading systems, it could not be used to apply the preflexing loads, which were required to remain constant for a period of a week.

Screw jacks.--Four twenty ton capacity, screw jacks were used to provide the end reactions when the beams were loaded in the loading frame. Two jacks, welded to a one inch steel plate at the bottom, and bolted to stiffened structural steel sections at the top, were placed under each end of the beam.



LOADING FRAME-END VIEW

Figure 11



LOADING FRAME - SIDE VIEW

FIGURE 12

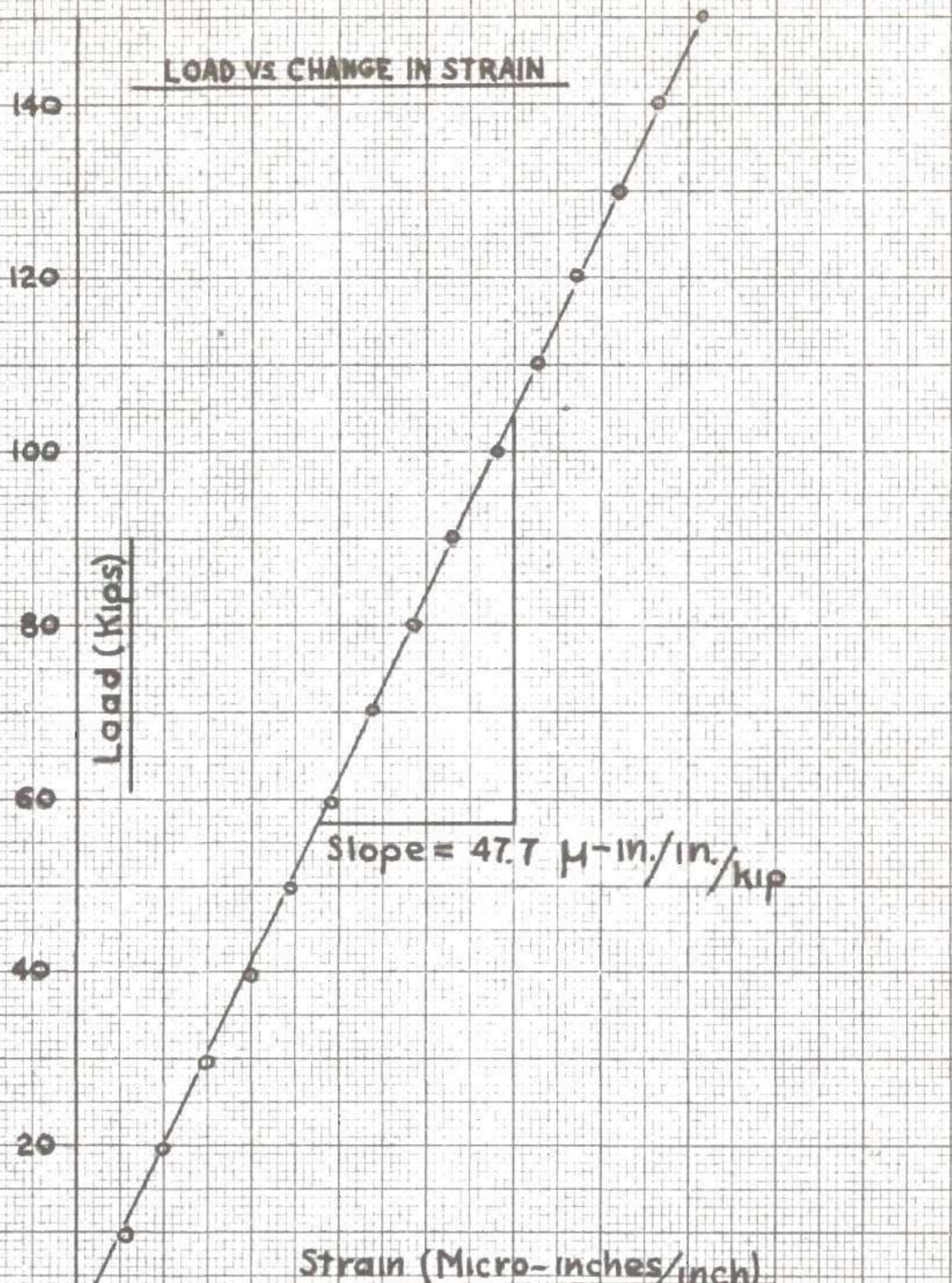
These screw jacks provided end reactions during the load tests, but they applied the preflexing load while the beams were being constructed. The screw jacks were used to apply the preflexing loads because they could be depended upon to provide a constant load for an extended period of time.

Proving ring.--A circular steel proving ring was used to measure the loads applied to the beams in the loading frame. The top of the proving ring was threaded to the piston of the hydraulic jack. The bottom of the ring rested on the distribution beam. The distribution beam transferred the single central load from the hydraulic jack to the two third points of the test beam.

Four SR-4 electrical resistance strain gages were mounted at 90 degree intervals around the inside circumference of the proving ring. These gages were wired into an external bridge circuit which resulted in increased sensitivity of the circuit. The ring was calibrated by recording the strains observed when the ring was subjected to known compressive loads in a two hundred thousand pound capacity universal testing machine. The calibration curves for this proving ring are shown in Figure 13.

Strain Measuring Equipment

The purpose of this research was to investigate the validity of the elastic theory in predicting the stresses which would occur in a preflexed beam. For this reason it was necessary to determine steel and concrete stresses at various points on the cross section of the beam at each stage in the preflexing process. Electrical resistance strain gages of the Baldwin SR-4 type were used to measure both steel and concrete strains. In addition, internal stress gages, for measurement of internal strains in



PROVING RING CALIBRATION CURVE

Fig. 13

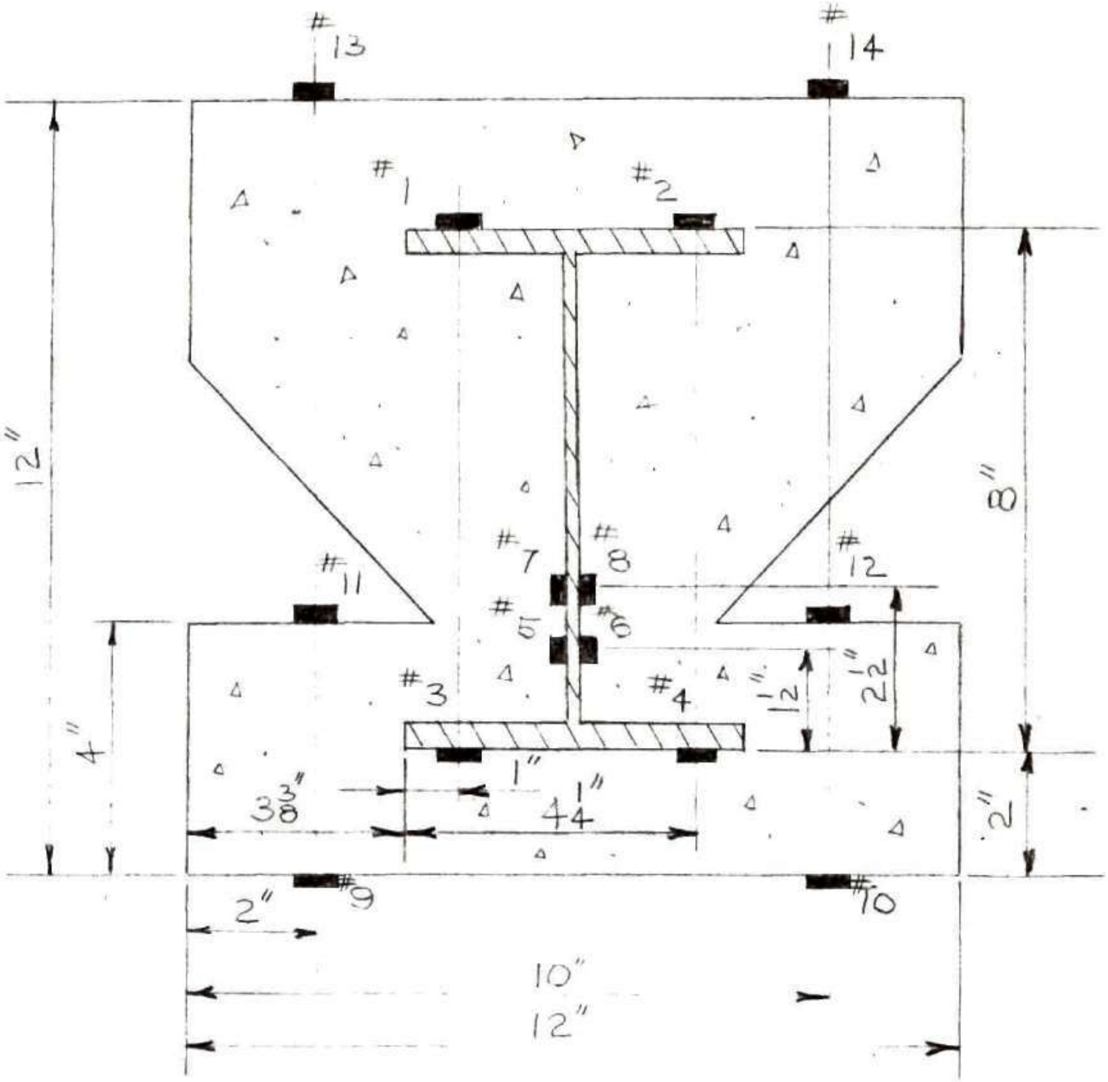
the concrete, similar to a type developed at Massachusetts Institute of Technology (10), were developed, but were eventually discarded, due to inability to eliminate drift and to properly waterproof the gages.

Measurement of steel strains.--Strains were measured on one cross-section of each of the three steel beams used in the tests. This cross section was located in the center, between the third point loads on the beam. Bonded electrical resistance strain gages, manufactured by Baldwin-Lima-Hamilton Corp., and commonly called "SR-4 strain gages", were used to measure steel strains throughout. On one beam, type A-1 gages were used, and on the other two beams, type A-11 gages were used. Eight gages were applied to each steel beam, two on the top flange, two on the bottom flange, and four on the web, as shown in Figure 14. These gages were applied in pairs, one gage of each pair on one side of the beam, and one on the other side. This was done so the results of each pair of gages could be averaged and strains due to torsion eliminated.

Measured steel strains were converted to stresses by multiplying the strains by the modulus of elasticity, E, of the steel. The value of E was assumed to be 30×10^6 p.s.i.

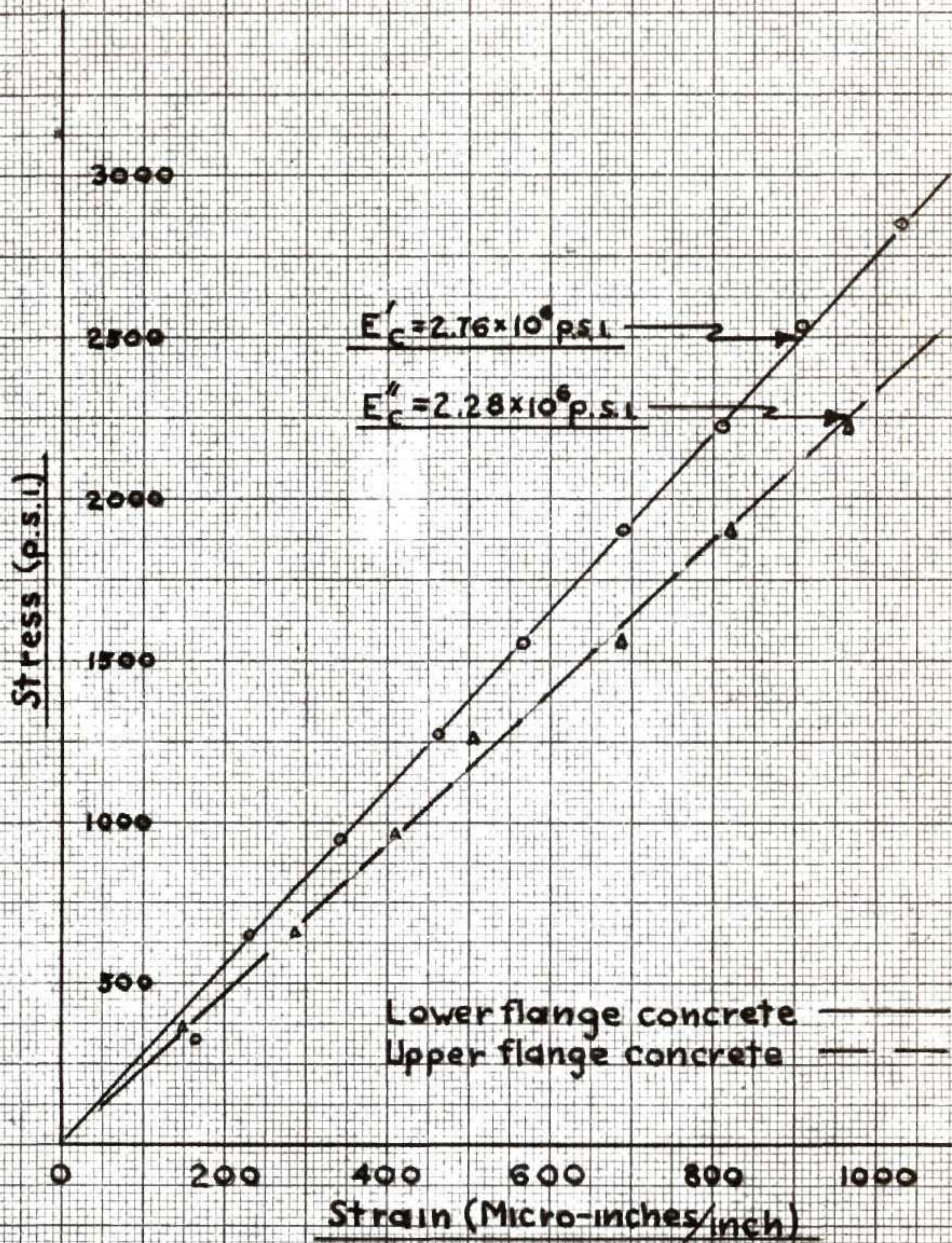
Measurement of concrete strains.--Concrete strains were measured on each beam at the same cross section as the steel strains were measured. Baldwin SR-4 strain gages were used exclusively for this purpose also. Six gages, in pairs, were applied to the concrete, four on the lower flange, and two on the upper flange. These gages were applied in pairs on the concrete, as they were on the steel, to eliminate the effects of torsion. See Figure 14 for the location of gages on the concrete.

Strains measured in the concrete were converted to stresses by com-



STRAIN GAGE LOCATION ON BEAM CROSS SECTION

Figure 14



BEAM CONCRETE – STRESS vs. STRAIN

Fig. 15

paring the strains with the stress-strain curve obtained from special compressive test cylinders made of similar concrete. These cylinders were two inches in diameter and eight inches high. They had two SR-4 type A-11 strain gages diametrically placed at the center of the cylindrical surface. A series of known loads was applied to each of these cylinders through a small proving ring, and the strain indicated by each strain gage was averaged with that indicated by the other gage on the same cylinder. From these data, a stress-strain curve was plotted for the mix used in the lower flange of each beam, and for the mix used in the upper flange of each beam. (See Figure 15)

Internal stress measurement.--It was originally planned to measure internal concrete stresses directly by using an internal stress gage for cementitious material, similar to those recently developed at Massachusetts Institute of Technology (10). These gages, as made for these tests, consisted of first one, and then two, Baldwin SR-4 type A-7 gages mounted on a hollow cylinder of high strength steel. This first cylinder was placed inside a second one, and tightened to it by a threaded cap. The space between the cylinders was filled with wax to prevent the entry of water.

The entire gage was embedded in a concrete cylinder, two inches in diameter and eight inches long. The gages were calibrated by applying a known load to the cylinder and recording the indicated strain. After calibration the gages were broken out of the concrete test cylinders and were to be embedded in the upper and lower concrete flanges.

Only two of the five gages prepared were placed, one in each of the lower flanges of the first two beams constructed. Initial readings were made with the first stress gage placed. The temperature-compensating gage, and the gage placed in the lower flange of beam number two both failed to register properly when tested. Because of this failure, use of the internal stress gages was discontinued. It was felt they were too delicate to function properly after having had concrete placed around them.

Deflection Measuring Equipment

It is claimed by Baes and Lipski (11) that one of the most important advantages of the "Preflex" method is that deflection caused by working loads is greatly reduced, when compared with the deflection caused by the same working load on a non-preflexed beam of equal dimensions and materials. In order to test the validity of this claim, beam deflections were measured with care during all phases of the tests.

When the plain steel beams were first loaded, in preparation for placing concrete around the lower flange, beam deflection could not be measured directly because the forms for holding the concrete surrounded the lower flange. At the same time, the distribution beam covered the center third of the top flange, and beam deflection could not be directly measured there either. It was necessary to weld quarter inch rods to the top flange, these rods projecting horizontally beyond the form used for the lower flange. Ames dials reading to 0.001" bore against the rods, and in this way the deflection was measured. The rods were used to measure beam deflection when the beams were loaded for preflexing, and when they were unloaded after the lower flange concrete had cured.

When the completed beams were load tested, deflections were measured directly. Ames dials located at the center and at the third points bore directly on the center of the lower flange.

Materials used.-- The physical properties of the steel and concrete used in the test beams are summarized in tables 1 and 2. Temperature and shrinkage steel composed of welded wire fabric, was used to reinforce each concrete flange. This fabric was made from 6 x 6-10/10 welded wire fabric.

Table 1 Physical Properties and Dimensions of Steel Beams Used in Tests

Size and Type Section	8 WF17
Length of Section	10'-0"
Material	Mayari "R" (See Table 3, Appendix "A" for heat record)
Producing Agency	Bethlehem Steel Co.
Yield Strength	56,960 p.s.i.
Ultimate Strength	94,610 p.s.i.

Table 2 Proportions and Physical Properties of Concrete Used in Tests

Lower Flange Concrete	
Average 7 day cylinder strength	4500 p.s.i.
Modulus of Elasticity	2.76×10^6 p.s.i.
Modular Ratio	10.9
Proportion of cement: sand: gravel by weight	1:0.94:1.54
Water/Cement Ratio	4.8 gal. per sack
Fineness Modulus of Sand	2.78
Maximum Size Aggregate	3/8"
Type Cement	High Early Strength
Upper Flange Concrete	
Average 7 day cylinder strength	5000 p.s.i.
Modulus of Elasticity	2.28×10^6 p.s.i.
Modular Ratio	13.1
Proportion of cement: sand: gravel by weight	1:1.27:2.1
Water/Cement Ratio	5.3 gal. per sack
Maximum Size Aggregate	3/8"
Fineness Modulus of Sand	2.78
Type Cement	High Early Strength

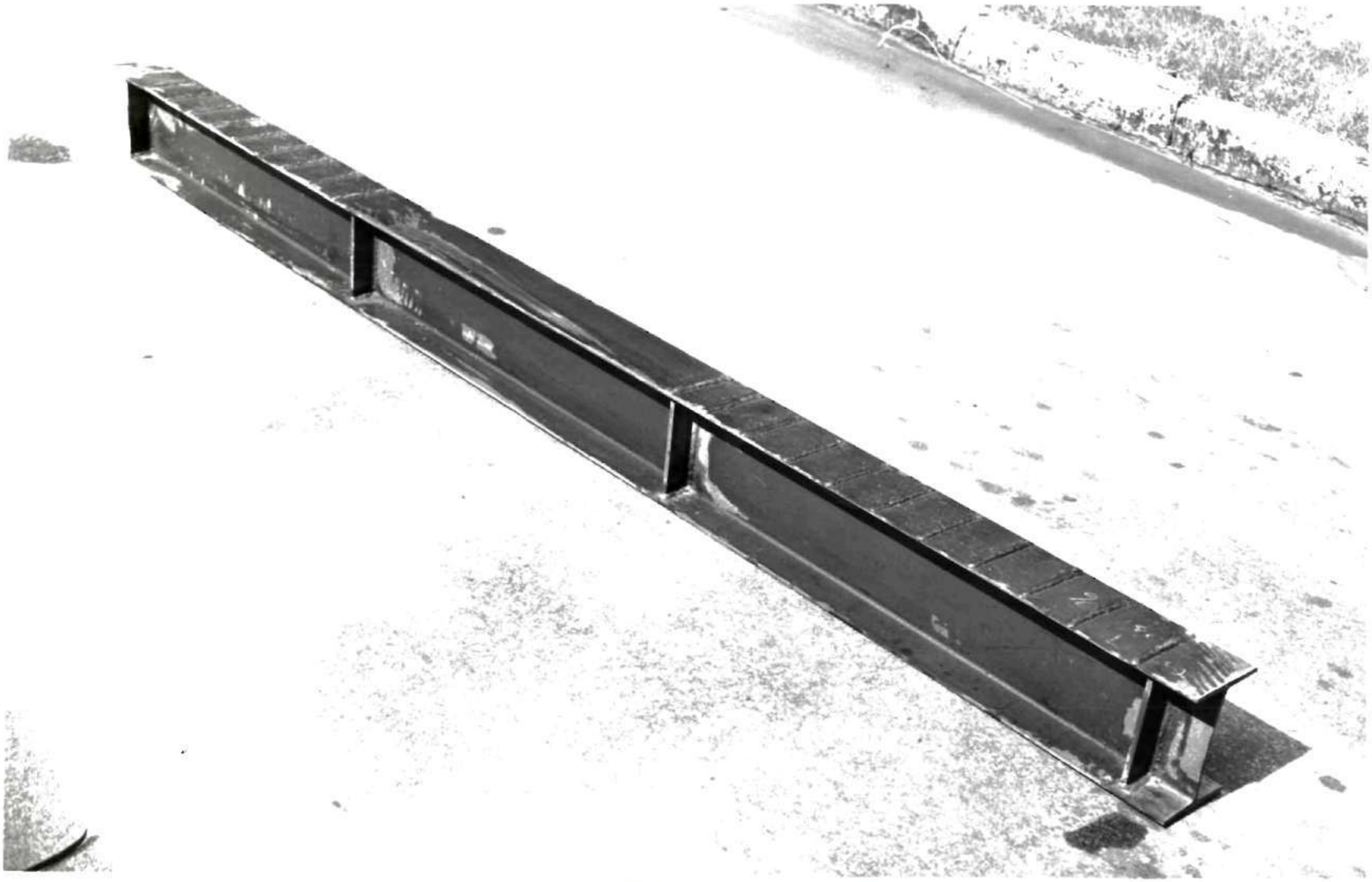


Figure 16. Modified Steel Beam

CHAPTER IV

PROCEDURE

The experimental work done in connection with this study of preflexed beams was divided into two phases. The first phase was the construction of the three test beams. The second phase was the load testing of the beams.

Construction of the Beams

Construction of the test beams began with modification of the steel. First, stiffeners made of 3/8" steel plates were welded on either side of the steel beams at each point where a concentrated load was to be applied. These points were at the ends and third points of the span, as shown in Figure 1. Second, beads of quarter inch weld were placed on the external surfaces of both the upper and lower flanges. These welds were made to roughen the flanges, and increase the bond between steel and concrete. No welds were placed in the middle third of the beam span, because, with the beam loaded at its third points, there would be little or no shear stress developed in the middle third. Figure 16 shows one of the steel beams after having been modified. Eight electrical resistance strain gages were then applied to the center of each beam as explained in Chapter III.

Control beam #1.--After the steel had been modified as described above, construction was begun on the control beam. A steel beam was placed in

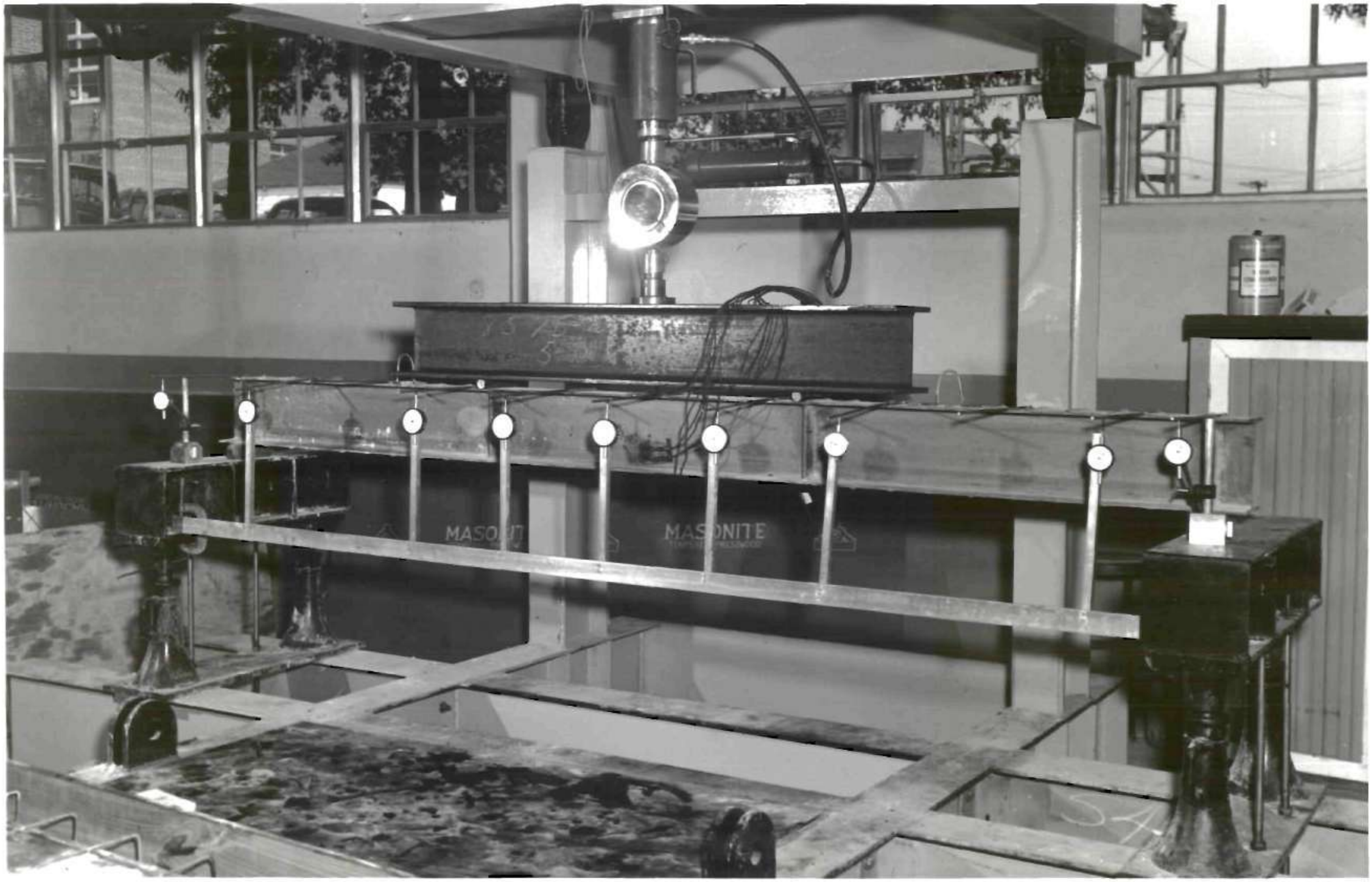


Figure 17. Apparatus for Applying Preflexing Load

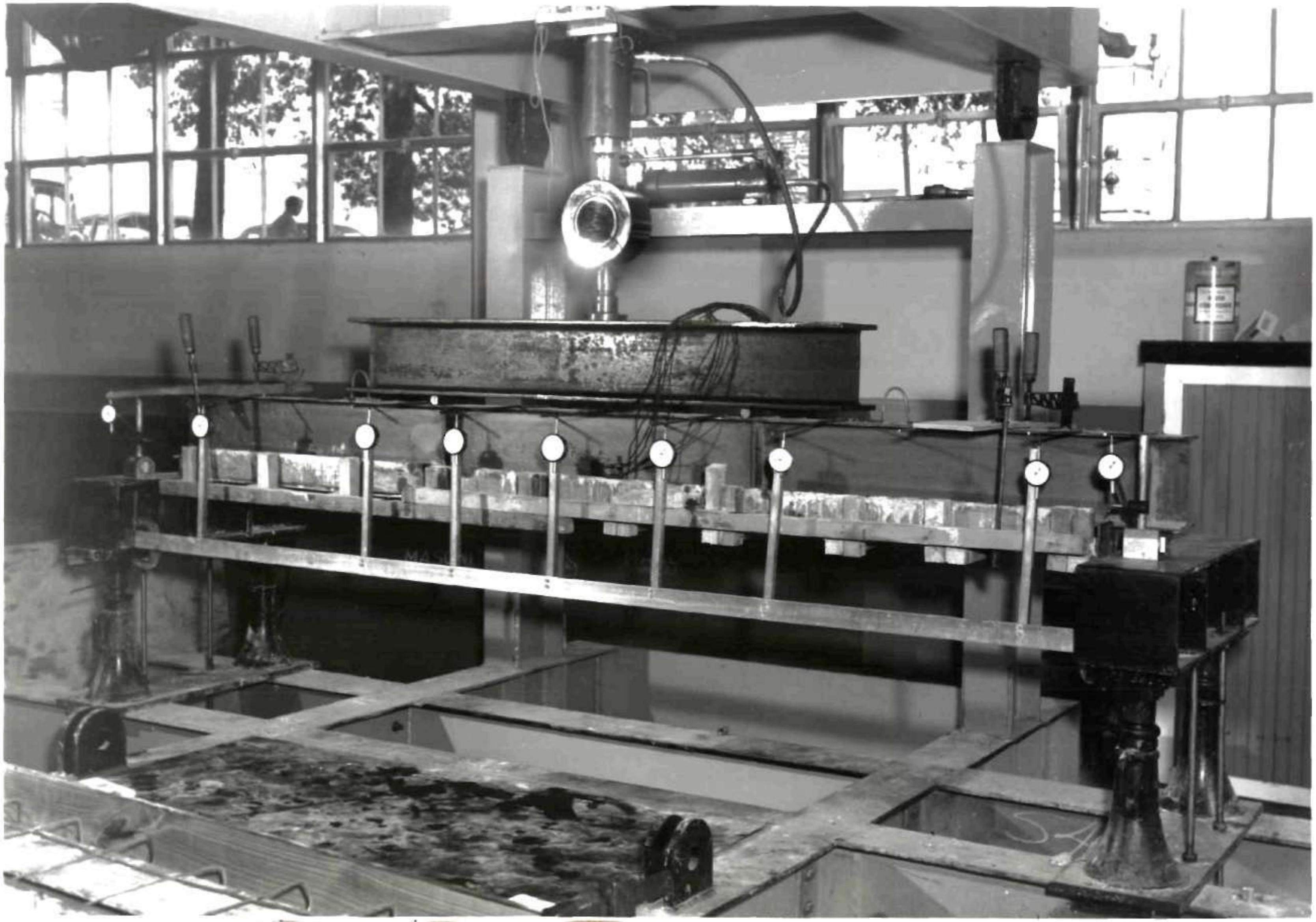


Figure 18. Location of Form for Lower Flange Concrete

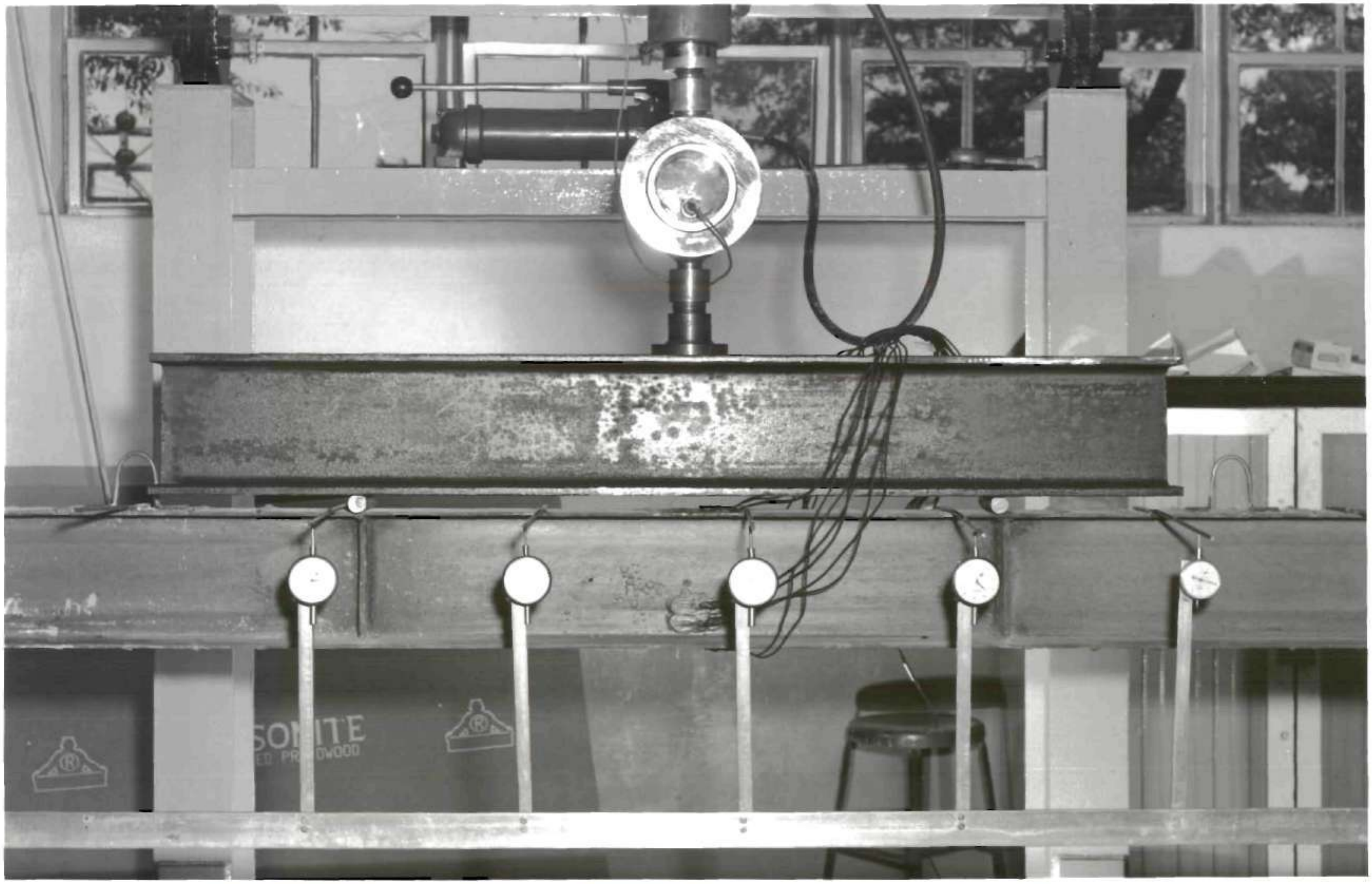


Figure 19. Close-up of Deflection Measuring Apparatus

the loading frame, as shown in Figure 17, but no load was applied. No deflection measurements were made during the construction of the control beam. With the beam in place, wire fabric was placed around the lower flange, and the lower flange form was clamped in place as shown in Figure 18. Concrete was placed in the form, vibrated, and allowed to cure for 24 hours. The forms were removed on the day after the pouring of the concrete. The beam was then taken out of the loading frame and set to one side for curing. This was accomplished by covering the concrete for a period of six days with burlap bags and empty cement sacks which were wet twice daily to keep them damp and thus insure proper curing. Two weeks after the lower flange concrete was placed, the upper flange concrete was placed and allowed to cure for five days in a similar manner. No effort was made to create any bonding between the two masses of concrete. The finished control beam had a cross section as shown in Figure 8. After the upper flange concrete cured, six A-11 strain gages were applied to the concrete, as shown in Figure 14.

Preflexed beam #2.--Preflexed beam #2 was constructed next, having identical dimensions with the control beam. The second steel beam was placed in the loading frame and Ames dials were placed to record the deflection of the beam as it was loaded, as shown in Figure 19. The preflexing load of 28,900 pounds was then applied to the third points of the beam, and the strain and deflection caused by its application measured. The load was applied by using the screw jacks at the ends of the beam. The railroad jacks were extended until the proving ring attached to the central hydraulic jack indicated that the proper preflexing load was being applied. The central hydraulic jack was not used to apply the preflexing load, because the load had to be held constant for seven days,

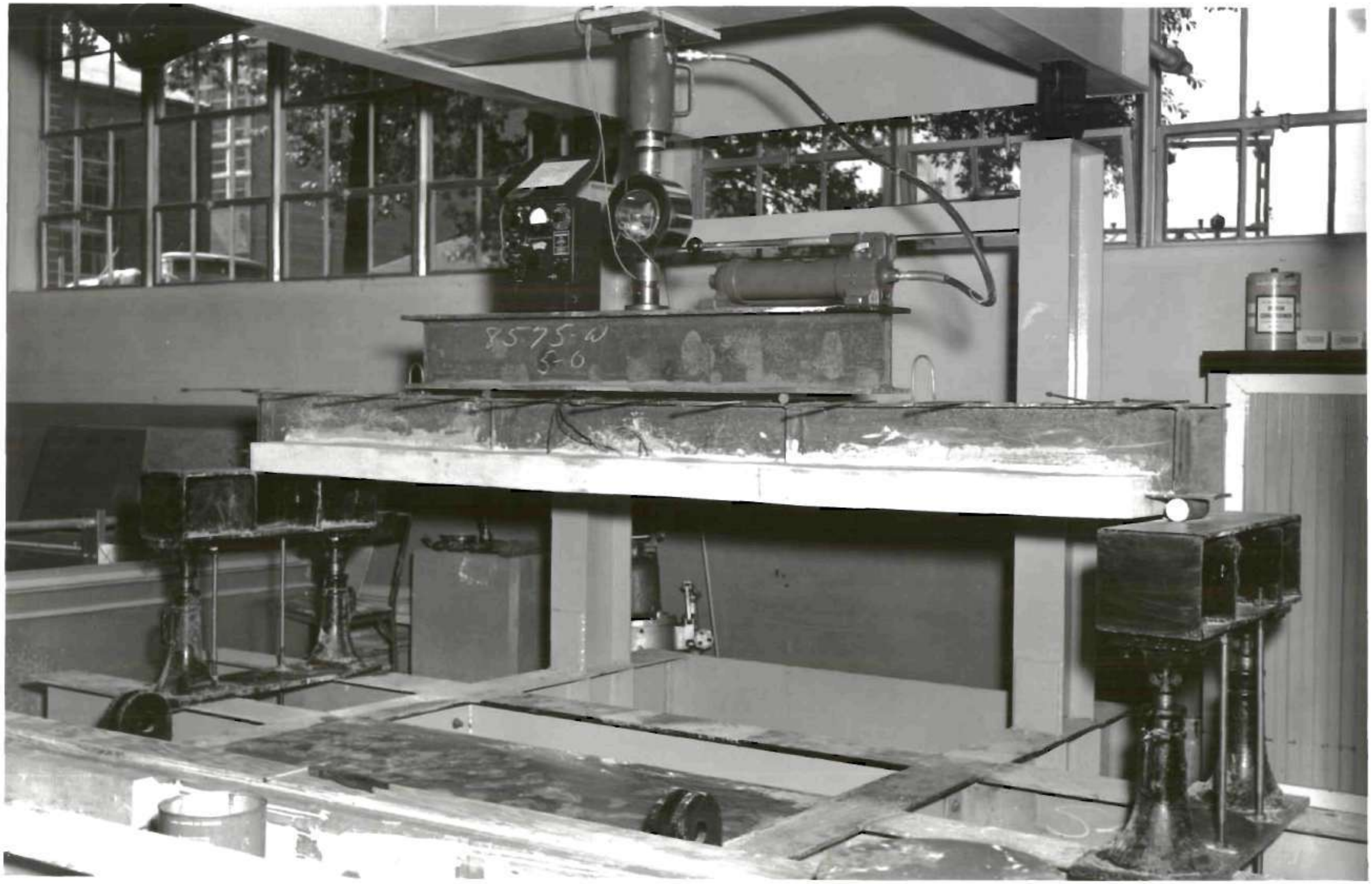


Figure 20. Lower Flange Concrete in Place

and the hydraulic jack would have leaked enough to vary the load substantially. With the preflexing load held constant, wire mesh was placed around the tension flange, and the form clamped in place, as shown in Figure 18. Then the lower flange concrete was placed in the form and allowed to harden for 24 hours.

After 24 hours the form was removed, and the green concrete was cured under wet burlap and cement sacks for six days. While the concrete was curing, four A-11 strain gages were applied to it, as shown in Figure 14.

Seven days after the tension flange concrete was placed, the preflexing load was removed. Changes in strain caused by removing the preflexing load were measured in both the steel and in the concrete, and the upward deflection of the beam was measured. Strains were measured at intervals after the preflexing load was removed to determine the amount of prestress loss.

Twelve days after the lower flange concrete was placed, the upper flange concrete was placed, and allowed to cure for five days. After the concrete cured, two A-11 strain gages were applied to it.

Preflexed beam #3.--Preflexed beam #3 was constructed exactly as preflexed beam #2, except that beam #3 had no concrete placed around its upper steel flange. The third steel beam was placed in the loading frame, and the preflexing load was applied. The lower flange concrete was placed, and allowed to cure for seven days while the preflex load was held constant. When the preflex load was removed, the concrete was prestressed. No further work was done on beam #3; it was load tested with no concrete covering on the upper flange of the steel.

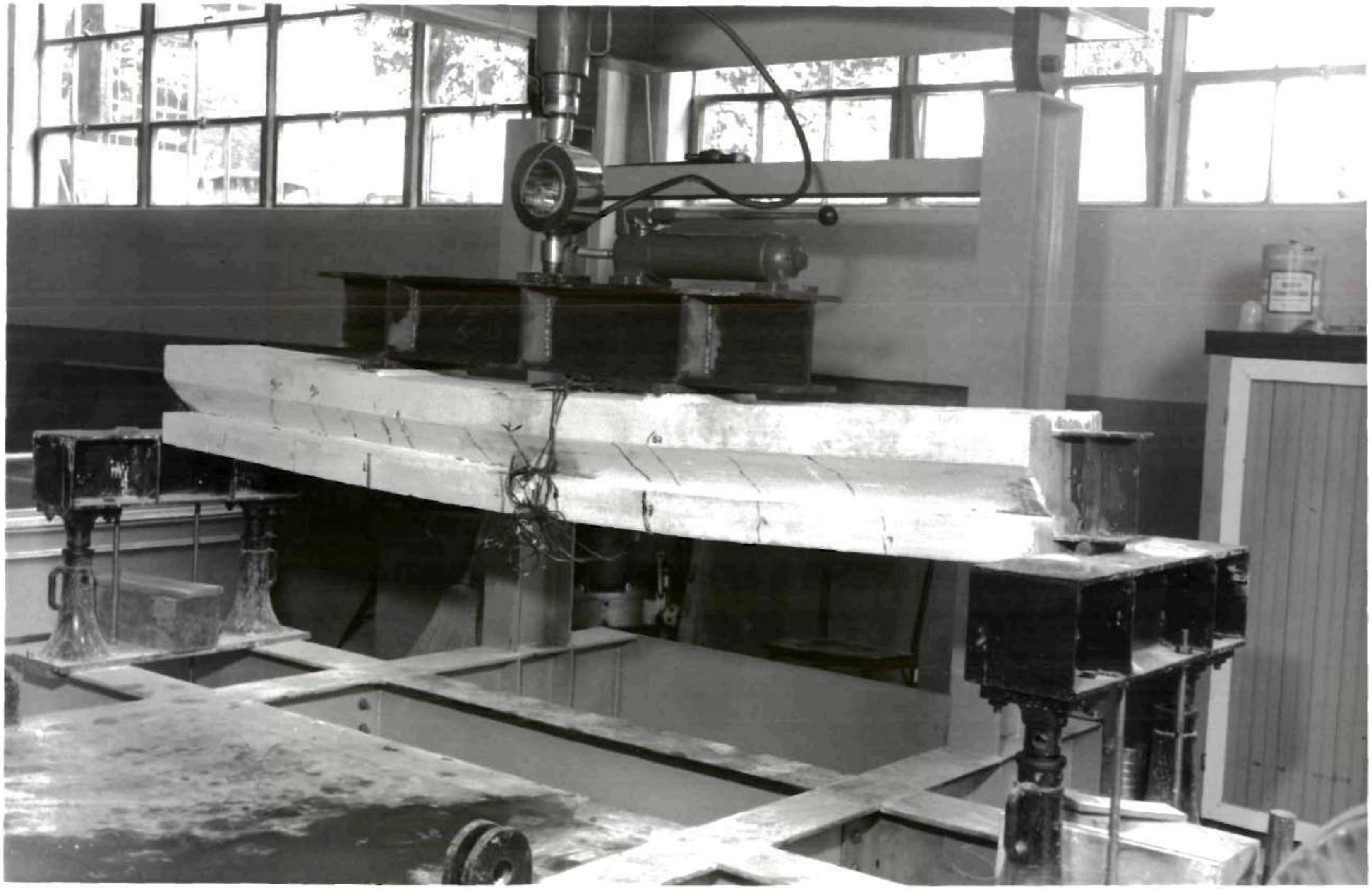


Figure 21. Ultimate Failure of Control Beam

Load Tests

After all the beams were constructed, each was load tested by applying concentrated loads at its third points. The loads were gradually increased until the ultimate load that the beam would carry was reached. Strain in both steel and concrete, and beam deflection, was measured for each increment of load.

Control beam #1.--Control beam #1 was placed in the loading frame, supported by the screw jacks, as shown in Figure 21. Ames dials were placed in contact with the lower flange to measure beam deflection. The test loads were applied by the central hydraulic jack, through the proving ring and distribution beam to the third points of the test beam. Three runs were made in the course of the tests on beam #1 before the ultimate load was applied.

On the first run, the hydraulic jack was powered by an electric centrifugal pump. It was difficult to accurately control the load applied by this pump, so the loads were applied in approximate instead of exact increments. Loads were applied in approximate increments of 10,000 pounds total load, or 5000 pounds load applied to each third point. Loads were always referred to as the total load applied to the beam. On the first run, loads were applied in 10,000 pound increments up to 60,000 pounds, and strain and deflection measurements were made for each increment of loading. The load at which successive cracks appeared in the concrete was noted as the test progressed. After the 60,000 pound load had been applied, all loads were removed, and the permanent deflection and strain were measured.

After the permanent set caused by the first run was measured, the second run was begun. During the second, and all subsequent runs, the hydraulic jack was powered by a hand pump. Loads were applied in exactly 10,000 pound increments until 60,000 was reached, and strains and deflections were measured at each increment. Again, after 60,000 pounds was reached, all loads were removed, and permanent strain and deflection read.

After the permanent set caused by the second run was measured, the third and final run was begun. Load was applied in 10,000 pound increments until the ultimate load was reached.

Preflexed beam #2.--Preflexed beam #2 was tested similarly to beam #1. The beam was placed in the loading frame, supported by the screw jacks. The load was applied to the third points of the beam by the hydraulic jack powered by the hand pump. Three runs were also made on the test of beam #2, but in a different manner from those made on beam #1.

In the first run, loads were applied to the beam in exact increments of 5,000 pounds, up to 25,000 pounds, and strain and deflection measurements were made at each increment. After the 25,000 pound load was applied, the beam was unloaded and permanent strain and deflection were measured.

After the permanent set caused by the first run was measured, the second run was begun. The load was immediately increased to 25,000 pounds and then by 5,000 pound increments to 50,000 pounds. The beam was then unloaded, and the permanent set caused by the second run measured.

In the third run, the load was immediately increased to 50,000 pounds, and then increased in 5,000 pound increments until the ultimate load was reached.

Preflexed beam #3.--Beam #3 was tested in the same manner as beam #2, except that the maximum load applied in each run was smaller. In run #1, the load was applied in 5,000 pound increments until 20,000 pounds was reached, and then the beam was unloaded. In run #2 the beam was immediately reloaded to 20,000 pounds and then loaded in 5,000 pound increments up to 40,000 pounds, after which it was unloaded. In run #3, the beam was immediately reloaded to 40,000 pounds, and then loaded in 5,000 pound increments until the ultimate load was reached.

CHAPTER V

RESULTS OF TESTS

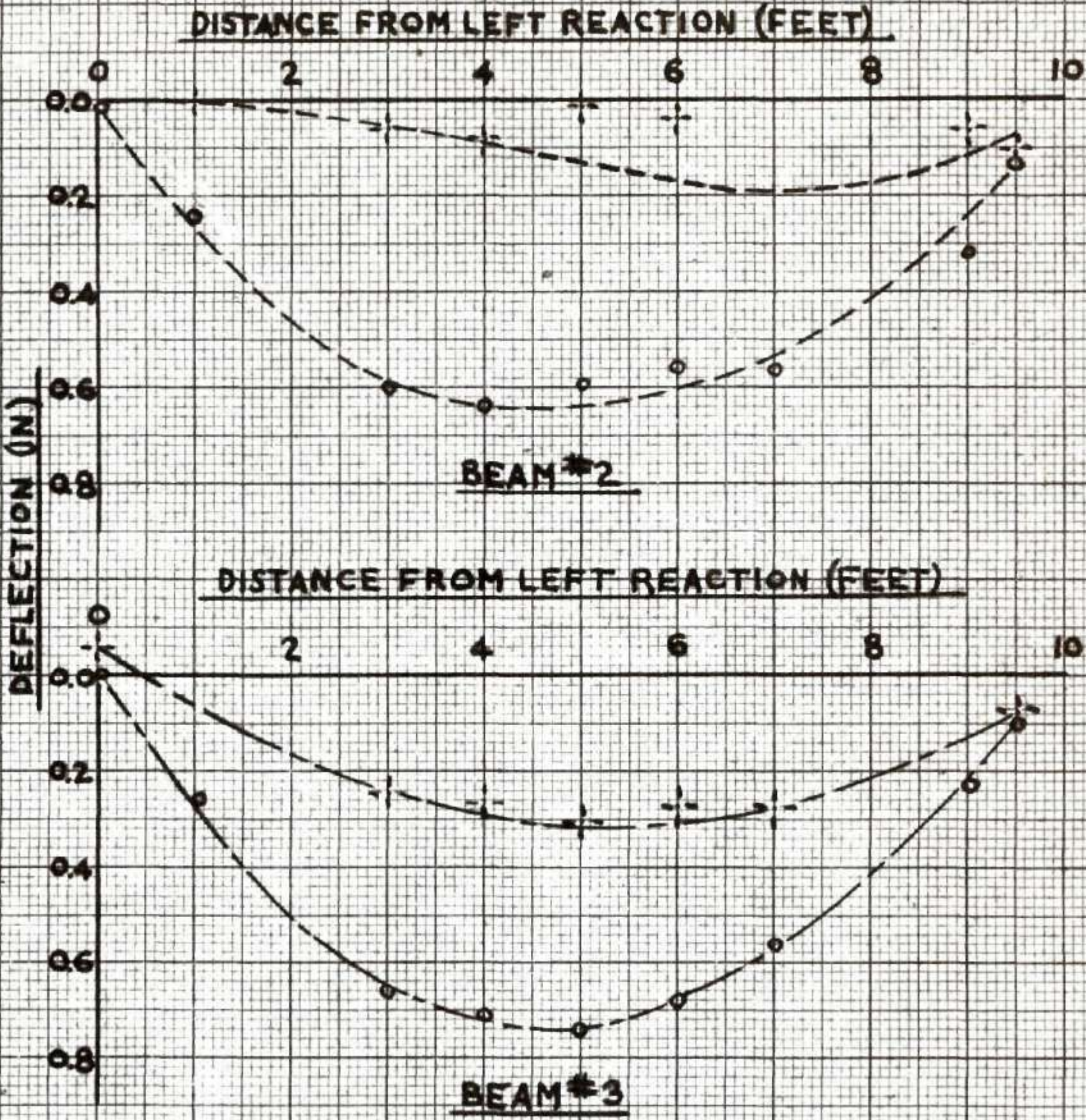
The results of the tests on the beams were divided into two groups. The first group was composed of results of tests made while the beams were being preflexed. The second group was composed of results of load tests made on the preflexed beams and the control beam after they were constructed.

In the first group of results, three problems in the construction of preflexed beams were studied. Deflections of the beams before and after preflexing were compared, measured stresses were compared with theoretical stresses computed by the elastic theory, and measured prestress loss was compared between the two preflexed beams.

In the second group of results, five phases of beam action were studied. Deflections were compared between preflexed and non-preflexed beams, cracking loads were compared, strains at various points in each beam were compared, measured stresses were compared with stresses computed by the elastic theory, and the ultimate loads of each beam were compared.

Results of the Preflexing Operation

Both deflections and strains were measured on the two beams which were prestressed by preflexing. Strains were measured across the center cross-section of each beam, and these strains were converted to stresses



DEFLECTIONS CAUSED BY APPLICATION OF PREFLEXING LOAD

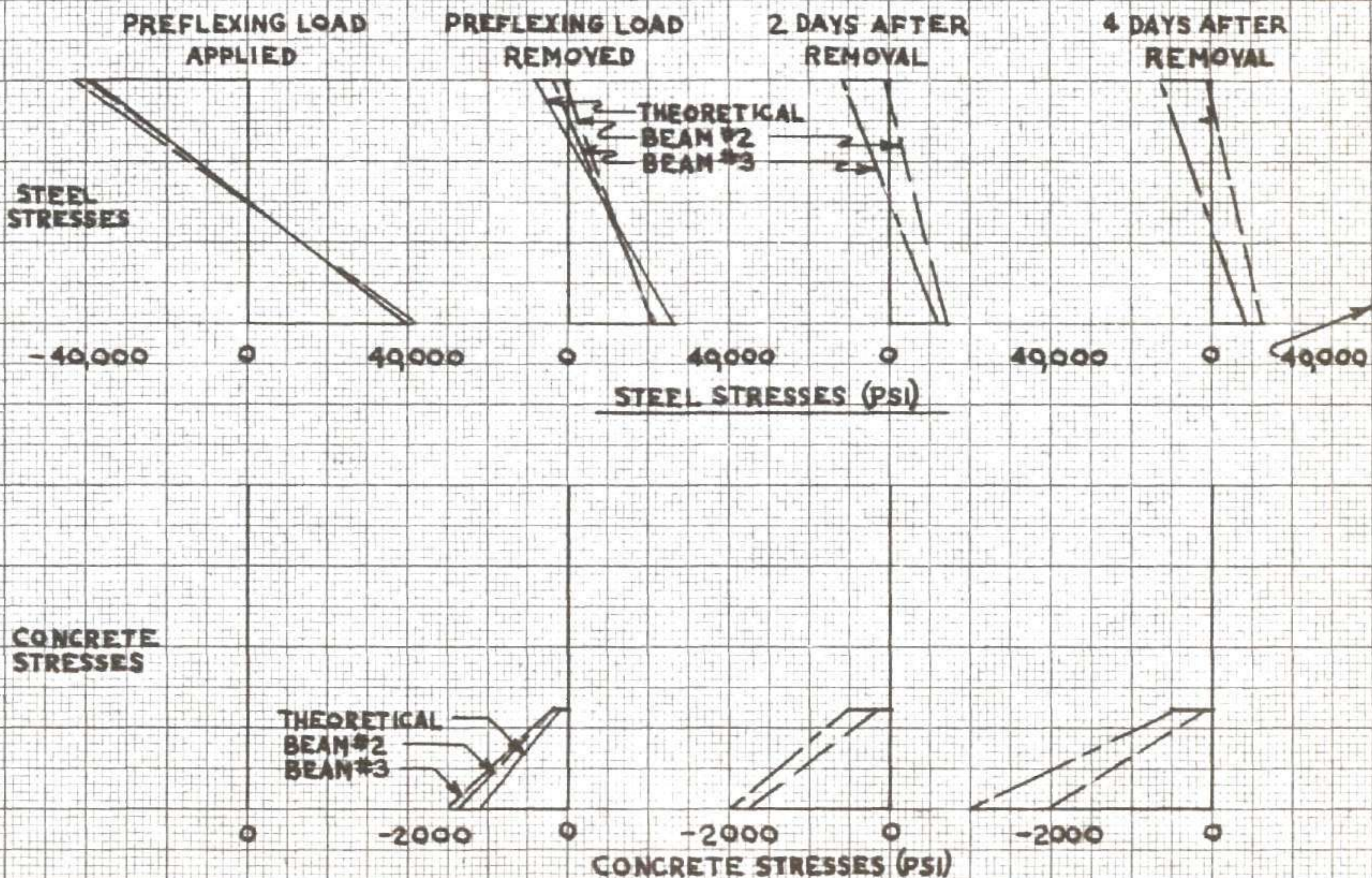
Fig. 22

in both steel and concrete.

Deflection.--When the plain steel beams were loaded with the preflexing load of 28,900 pounds, deflections were measured as described in Chapter III on both beams. After the lower flange concrete had been placed and cured, deflections were again measured when the preflexing loads were removed. The difference between these two readings was the initial permanent deflection caused by prestressing the lower flange concrete. The readings on the beam #2 were erratic and will not be discussed here. A plot of these readings appears on Figure 22, if the reader wishes to inspect them. On beam #3, the center deflection of the plain steel beam under the preflexing load was 0.75". When the preflexing load was removed, the upward deflection of the beam was 0.45", leaving an initial permanent deflection of 0.30". This permanent deflection was caused by prestressing the lower flange concrete. Deflections caused in beam #3 by the preflexing operation are plotted in Figure 22.

No readings were made to measure the loss in initial permanent deflection which was caused by prestress loss during the time between the removal of the preflexing load and the beginning of the load tests. From strain readings taken during this interval, the loss in deflection was probably appreciable.

Measured stresses.--Strain readings, as described in Chapter III, were made on the cross section at the centerline of each beam while it was being preflexed. To transform these strain readings to stresses, the strains were multiplied by an appropriate modulus of elasticity. The modulus for the steel was assumed as 30×10^6 p.s.i., and the modulus for lower flange concrete was determined by tests as 2.75×10^6 p.s.i. (See Figure 15)



STRESSES IN BEAMS CAUSED BY PREFLEXING

Fig. 23

From the strain readings, maximum stress in the plain steel beams, when the preflexing load of 28,900 pounds was applied to the third points, was 42,400 p.s.i. in the upper flange, and 40,500 p.s.i. in the lower flange. Both of these maximum stresses occurred in beam #3. Immediately after the preflexing load was removed, the maximum upper flange steel stress was reduced to 3,700 p.s.i. compression, and the maximum lower flange steel stress was reduced to 21,300 p.s.i. tension. Prestress in the lower flange concrete after removal of the preflexing load varied from 365 p.s.i. compression, at the top of the flange, to 2950 p.s.i. compression, at the bottom of the flange. Steel and concrete stresses due to preflexing in beams #2 and #3 are summarized in Figure 23.

Loss of prestress.--Measurements of the amount of prestress loss which occurred during the first week after removal of the preflexing loads were inconclusive. Strain measurements on beam #3 were erratic, and are not considered in this report. The strain measurements on beam #2 appear to be reliable, and the discussions of prestress loss in this report are based on them. The results of only one test are quite inconclusive, and no conclusions are made regarding prestress loss.

On the basis of strain readings on beam #2, lower flange steel stress dropped from 20,800 p.s.i. immediately after unloading, to 12,700 p.s.i. six days later. This represents a prestress loss of 38.91 percent in six days. Concrete prestress loss was not computed, because the large strain in the bottom of the lower flange concrete indicated appreciable plastic flow, and stress was not proportional to strain. In Figure 23, prestress loss in both steel and concrete is summarized for beam #2. The excessive concrete stresses indicated are not correct, but they are proportional to the measured strains.

Results of Load Tests

Load tests were performed on each of the three beams constructed. Loads were applied in these tests in the same manner that the preflexing loads were applied, varying from zero to the ultimate load that the beams would carry. Strains and deflections were measured for each increment of load. Center line deflections of preflexed and non-preflexed beams were compared. Strains at various locations on the center line of each preflexed beam were compared with each other and with the strains on the non-preflexed beam.

Strains across the center cross section of each preflexed beam were converted to stresses by multiplying them by an appropriate modulus of elasticity. These stresses were then compared with theoretical stresses based on the elastic theory.

Deflections.--Deflections at the center and at the third points were measured for each increment of load on each of the three beams tested. Measured third point deflections were consistent with measured center deflections throughout the load tests. To eliminate duplication, only the center deflections are discussed in this report.

At the preflexing load of 28,900 pounds, which may also be considered the working load, the non-preflexed control beam, beam #1, had a measured center deflection of 0.35 inches. For a $9\frac{1}{2}$ foot span length, this deflection is equivalent to 1/326 of the span length. At the same load of 28,900 pounds, the first preflexed beam, beam #2, which was identical in size with the control beam, had a center deflection of 0.29 inches. This deflection was equal to 1/391 of the span length.

At this same load, the second preflexed beam, beam #3, which had no concrete placed around the upper steel flange, had a center deflection of 0.443 inches. This deflection corresponded to 1/258 of the span length. For comparison, the deflection of a plain 8 WF 17 at the preflex load was computed to be 0.511 inches, corresponding to 1/223 of the span length.

At a test load of 20,000 pounds, before the lower flange concrete on either of the preflexed beams had cracked, the difference in deflection was even more pronounced. For the control beam, beam #1, the center deflection was 0.242 inches. For beam #2, the center deflection was 0.154 inches, or 63% of that of the control beam. For beam #3, the center deflection was 0.261 inches or 108% of the control beam, and 170% of beam #2. Again for comparison, the computed center deflection of a plain 8 WF 17 was 0.351 inches, or 145% of the control beam and 134% of beam #3.

Results of the center deflection readings for all the load tests are summarized in Figure 24.

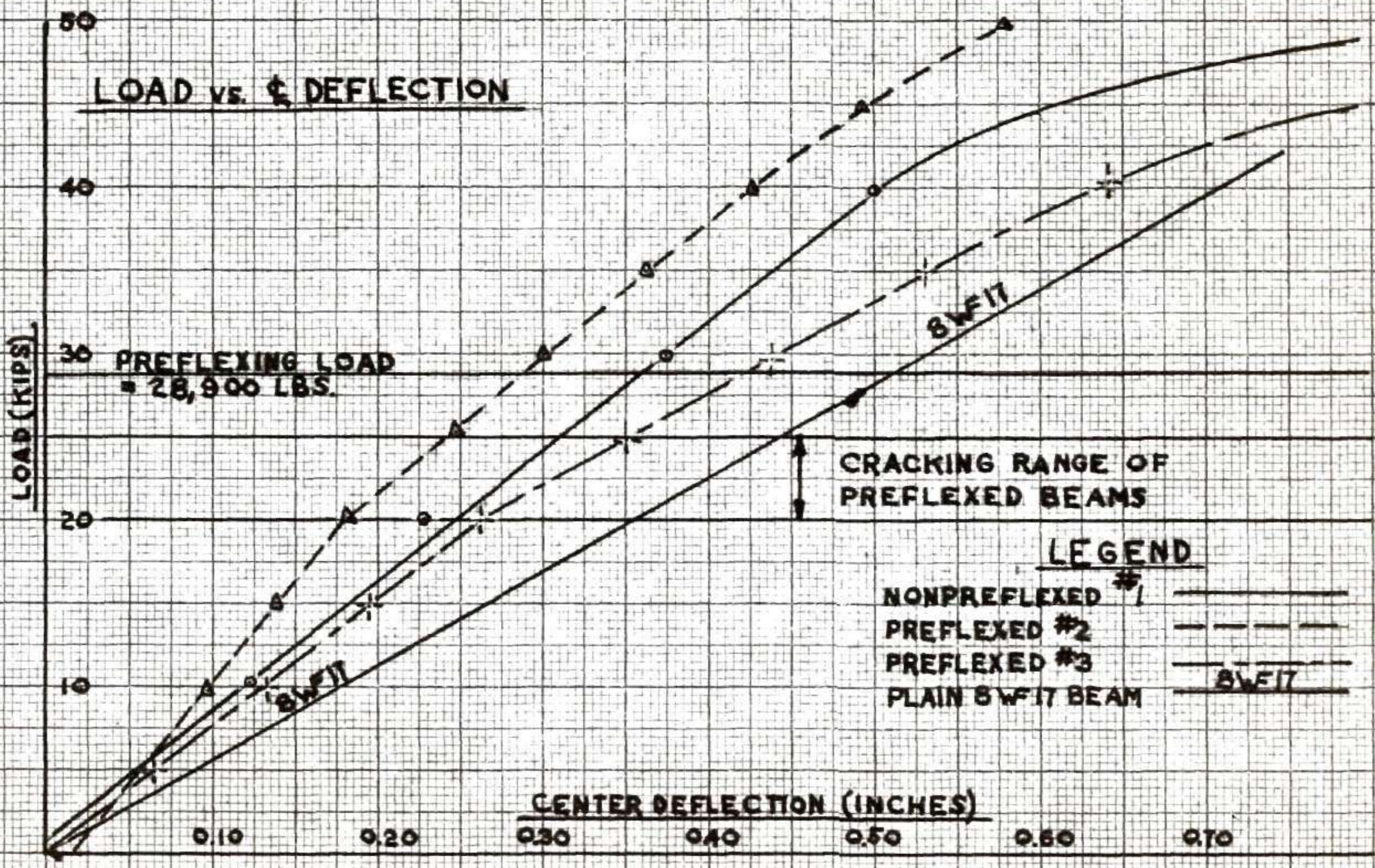
Cracking.--Careful notations were made of the loads at which initial tension cracks appeared in the lower flange concrete of all three beams. In the preflexed beams, the formation of tension cracks indicated that the effects of prestressing had been overcome. The gross section of concrete was no longer effective in resisting bending after the first cracks appeared. Only the steel and the net concrete in the uncracked portion were effective in resisting bending after cracking began.

In beam #1, the non-preflexed control beam, the lower concrete flange cracked completely through in three different places at less than 10,000 pounds load. Cracks #1,#2, and #3 had all appeared at that load.

As additional loads were applied, these initial cracks grew longer and wider, indicating that the neutral axis of the beam was continually moving upward. New cracks, ever nearer the ends of the beam, appeared as the loads were increased. After beam #1 was loaded with 50,000 pounds, the load was removed. All cracks tended to become smaller, but they failed to close completely, indicating that substantial permanent set had already taken place. Initial permanent deflection at the center of the beam was measured as 0.40 inches at this time.

In beam #2, the first evidence of cracking did not appear until more than 20,000 pounds had been applied. Cracking had become general by the time 25,000 pounds were applied, the lower flange having been cracked completely through in two places. At this load, however, cracking in beam #2 was much less severe than in beam #1. After the application of the 25,000 pound load, all loads were removed from beam #2. With removal of the load, both cracks in the lower flange closed completely. Permanent center deflection was measured as 0.017 inches. When the loading was continued past 25,000 pounds, the number of tension cracks increased, and the original cracks became longer.

In beam #3, cracking began at slightly more than 20,000 pounds and was again widespread at 25,000 pounds, with the lower flange cracked through in two places. This cracking was much less severe than that which occurred in beam #1 at the same load. The two cracks completely closed when the load was removed. After the removal of the load, the permanent deflection in the center was 0.010 inches. When the load test was continued, the cracks re-opened and they grew longer when additional load was applied.



DEFLECTION OF BEAMS DURING LOAD TESTS

Fig. 24

Strain.--Strains were measured during the load tests by means of electrical resistance strain gages. They were measured in both concrete and steel in the upper and in the lower flange of each beam.

Steel strain in the lower flanges was greatly influenced by the formation of cracks in the lower flange concrete. At loads less than the cracking load of 20,000 to 25,000 pounds, the rate of strain per unit of load was 147 micro-inches per inch per 1000 pounds in beam #2, and 160 micro-inches per inch per 1000 pounds in beam #3. No strain readings below the cracking load were made for beam #1, because of the low load at which cracking occurred. At loads greater than the cracking load, the rate of strain per unit of load increased abruptly. The rate of strain was 413 micro-inches per inch per 1000 pounds for beam #2, 473 micro-inches per inch per 1000 pounds for beam #3, and 390 micro-inches per inch per 1000 pounds for the non-preflexed beam, beam #1.

Concrete strain on the bottom of the lower flange was greatly affected by the cracking of the concrete, as was expected. At loads less than the cracking load, the rate of strain was 228 micro-inches per inch per 1000 pounds for beam #2, and 258 micro-inches per inch per 1000 pounds for beam #3. Again, no strain readings were made on beam #1 at loads below the cracking load. At loads greater than the cracking load, the strain was practically constant in lower flange concrete for all three beams.

Steel strain in the upper flanges was not affected by cracking of the lower flange concrete. The unit rate of strain was constant throughout the working range, up to 30,000 or 35,00 pounds load. The unit rate of strain was 183 micro-inches per inch per 1000 pounds for beam #1, 162 micro-inches per inch per 1000 pounds for beam #2, and 404 micro-inches per inch per 1000 pounds for beam #3.

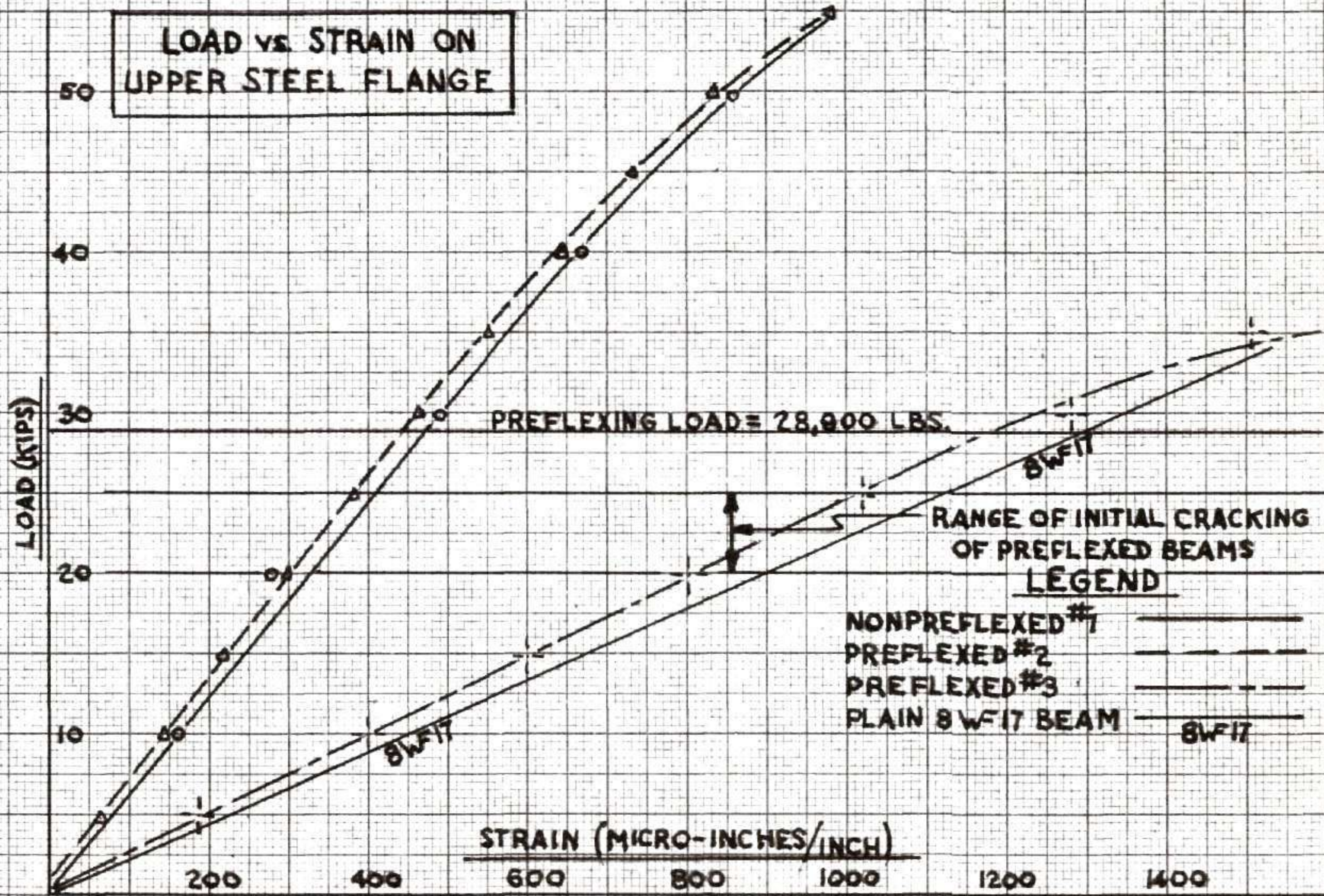
Concrete strain in the upper flange was slightly affected by cracking of the concrete in the lower flange. Below the cracking load the unit rate of strain was 150 micro-inches per inch per 1000 pounds for beam #2. Strain was not measured below the cracking load of beam #1. Above the cracking load the unit rate of strain was 255 micro-inches per inch per 1000 pounds for beam #2, and 288 micro-inches per inch per 1000 pounds for beam #1.

Strain measurements for all the load tests are summarized in the plots of load vs. strain in Figure 25 through Figure 28.

Measured stresses.--Within the range of loadings where strain varied linearly with applied load, measured changes of strain were converted to changes of stress by multiplying changes of the strain by an appropriate modulus of elasticity. This procedure is explained in Chapter III.

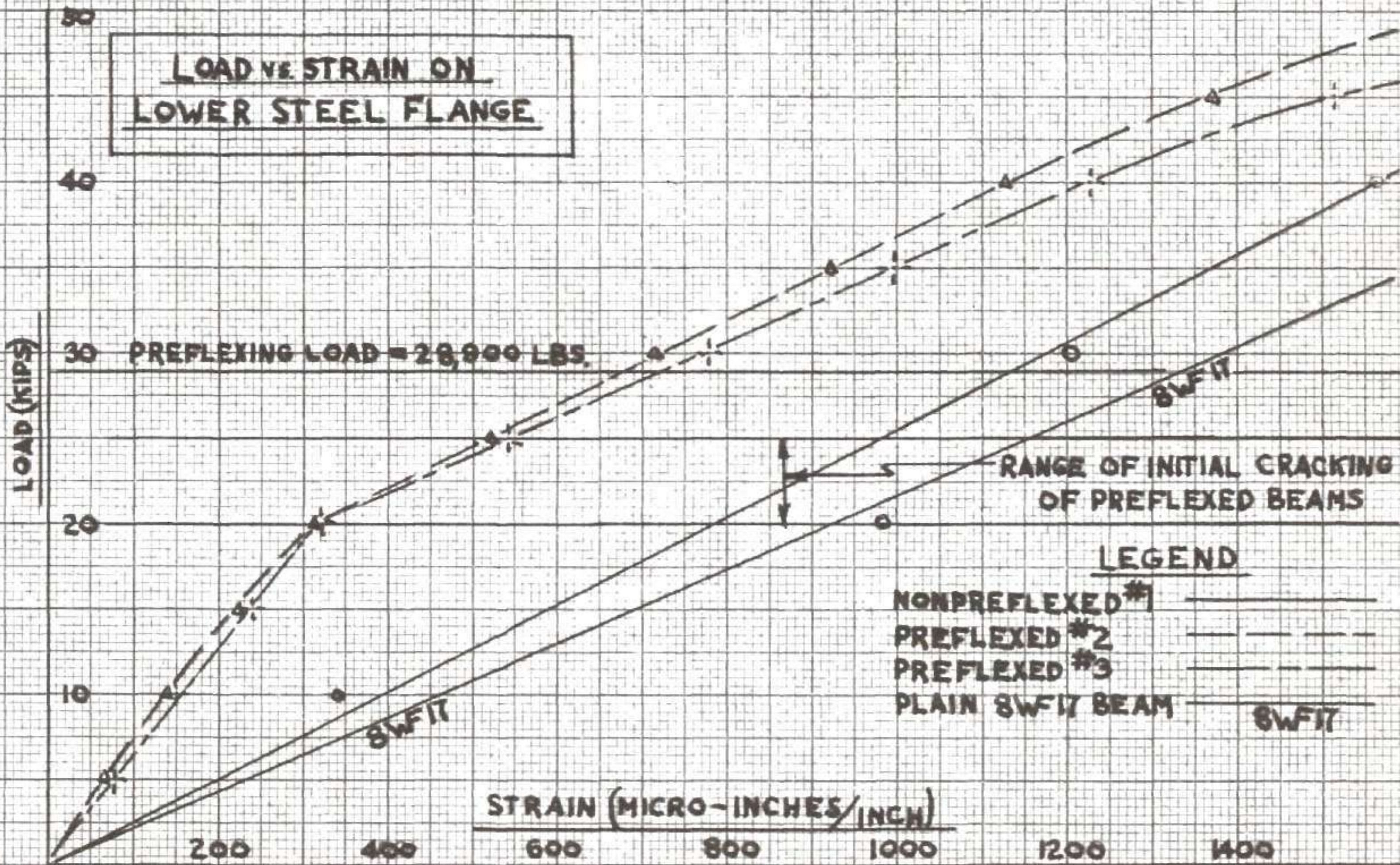
It was not possible to compute the initial stress or prestress magnitude in a beam from the initial strain-reading of a load test. Therefore, all stresses were assumed to be zero at the beginning of each load test, although they were known to be far from that condition in many instances. With this assumption of zero stress and strain as the initial condition for all the load tests, the measured strains and computed stresses at any load actually represent changes of strain and changes of stress for the indicated change of load. To measure the total strain or stress in a given beam under given load, it would be necessary to add the unknown initial strain or stress to the change in strain or change in stress recorded during the load tests.

Computed "changes in stresses", subsequently referred to as "stresses", were plotted in their proper positions on a cross section of



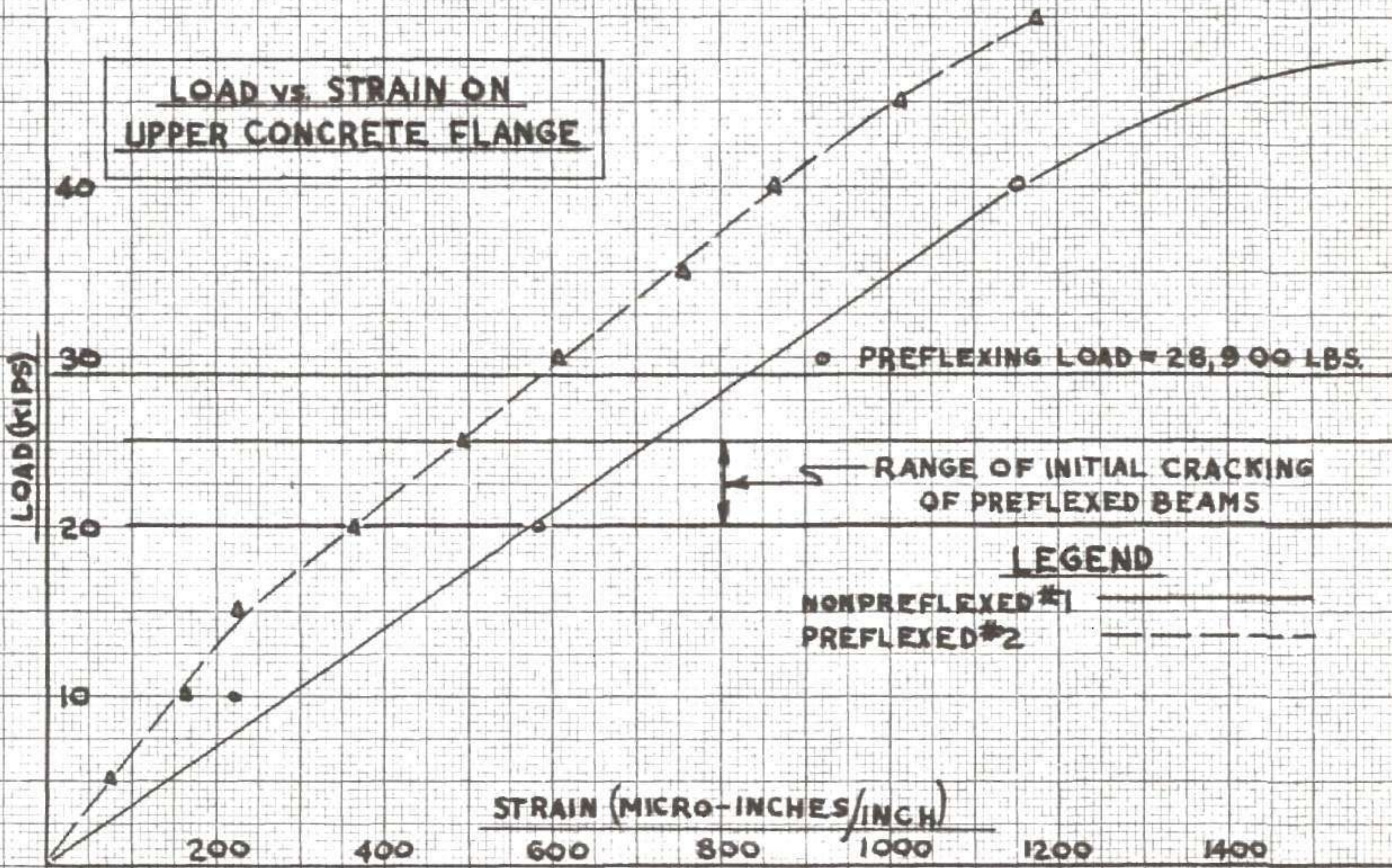
STRAIN IN UPPER STEEL CAUSED BY LOAD TESTS

Fig. 25



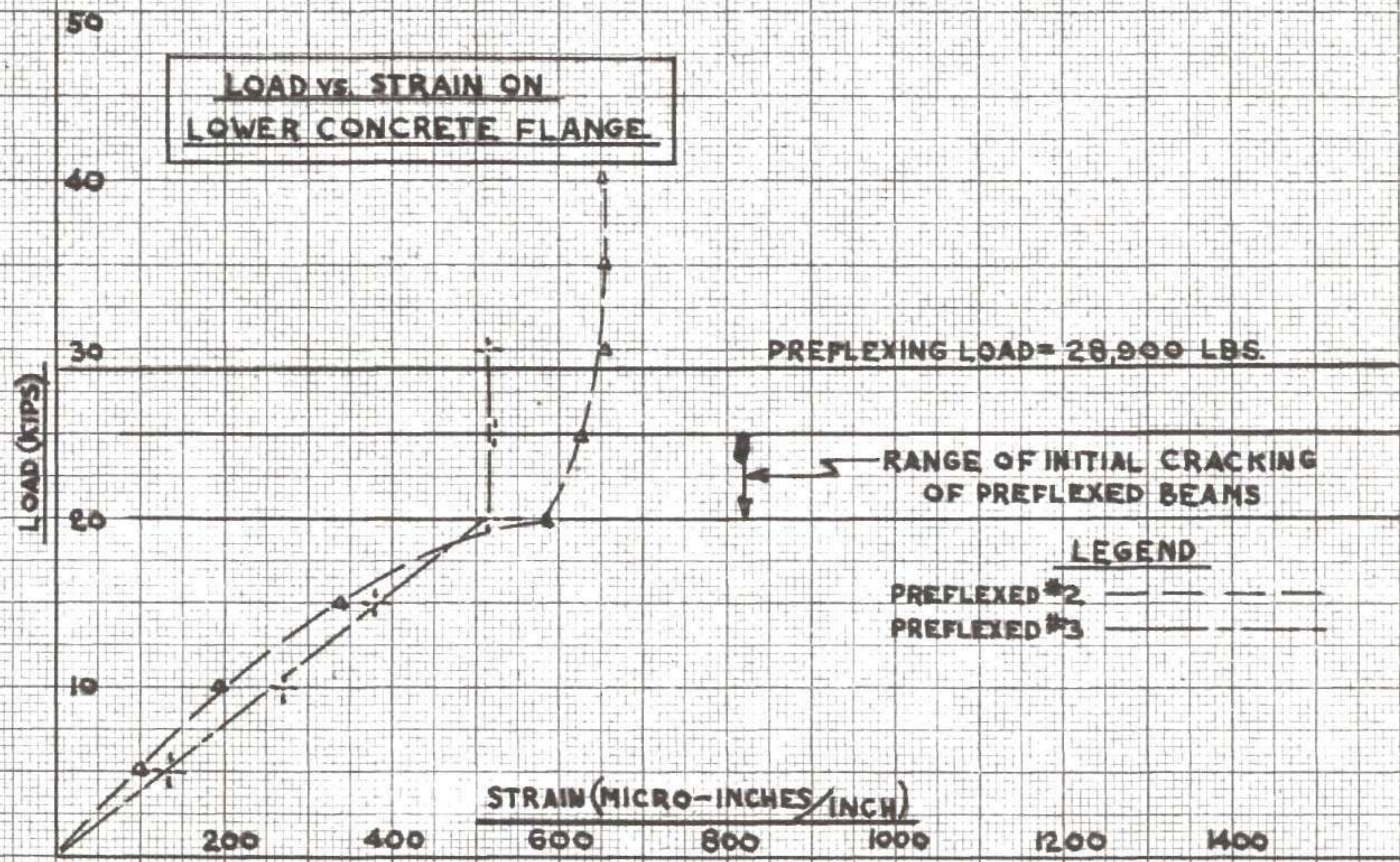
STRAIN IN LOWER STEEL CAUSED BY LOAD TESTS

Fig. 26



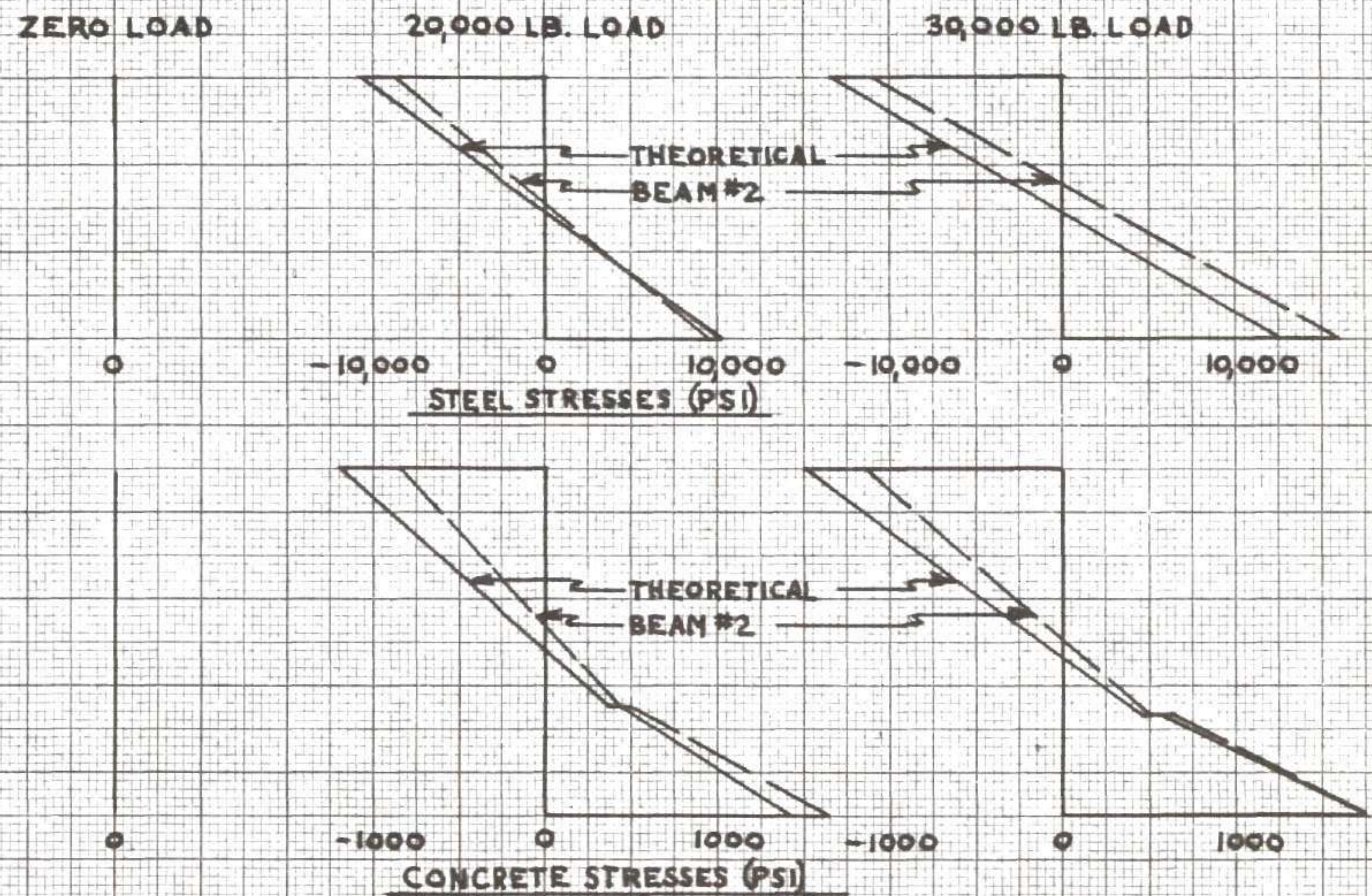
STRAIN IN UPPER CONCRETE CAUSED BY LOAD TESTS

Fig. 27



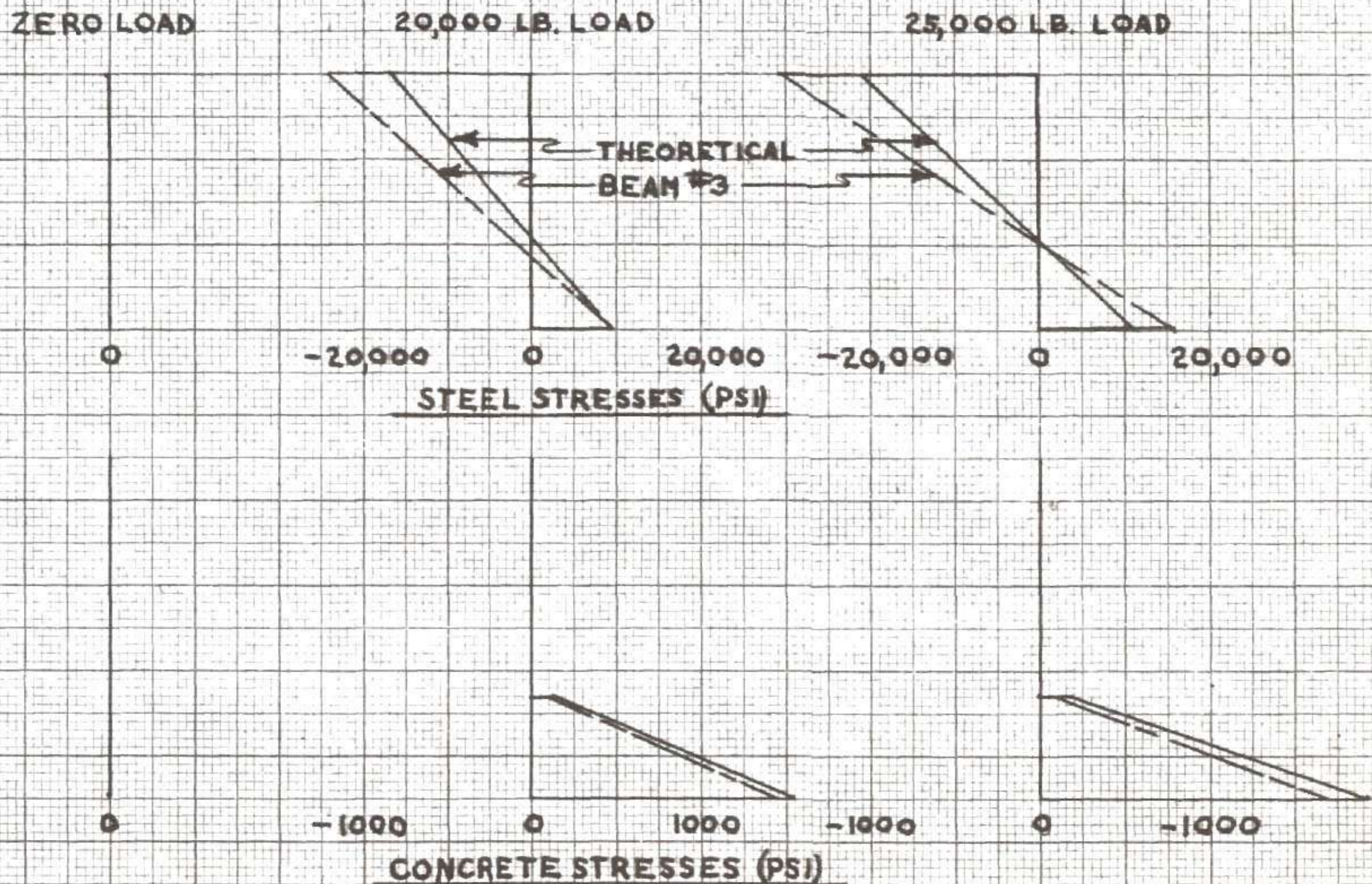
STRAIN IN LOWER CONCRETE CAUSED BY LOAD TESTS

Fig. 28



STRESSES IN BEAM #2 CAUSED BY LOAD TESTS

Fig. 29



STRESSES IN BEAM #3 CAUSED BY LOAD TESTS

Fig. 30

a beam. They were plotted for loads of zero, 20,000 pounds, and 25,000 pounds. The measured stresses, and the stresses predicted by the elastic theory are plotted and compared in Figures 29 and 30.

Ultimate Load Results

As the loads on the tested beams were increased, cracking became more severe. The cracks steadily extended upward, until at failure, they were within half an inch of the tops of beams #1 and #2. Initial failure in these two beams was a tension failure, but the final failure was a compression failure in the top flange. The area of compression concrete was steadily reduced as the beam deflected and the lower steel flange entered the range of plastic deformation. This reduction in area of the compression concrete continued until the concrete in the upper flange was stressed to its ultimate compressive strength, and final failure occurred. Figures 29, 30, and 31 show the type of failure which each beam experienced.

Prestressing had no effect upon the ultimate strength of the beams tested, nor upon their deflection at ultimate load. The ultimate load which beam #1 carried was 66,800 pounds, and center deflection at that load was 2 3/16 inches. The ultimate load carried by beam #2 was 66,700 pounds, with a center deflection of $2\frac{1}{4}$ inches.

Beam #3, which had no concrete covering the upper steel flange, failed in lateral buckling instead of in compression. The lower flange concrete could not provide sufficient lateral stiffness to the compression flange to prevent that type of failure. The ultimate load carried by beam #3 was 48,800 pounds.

The limit of proportionality of strain to load was not clearly

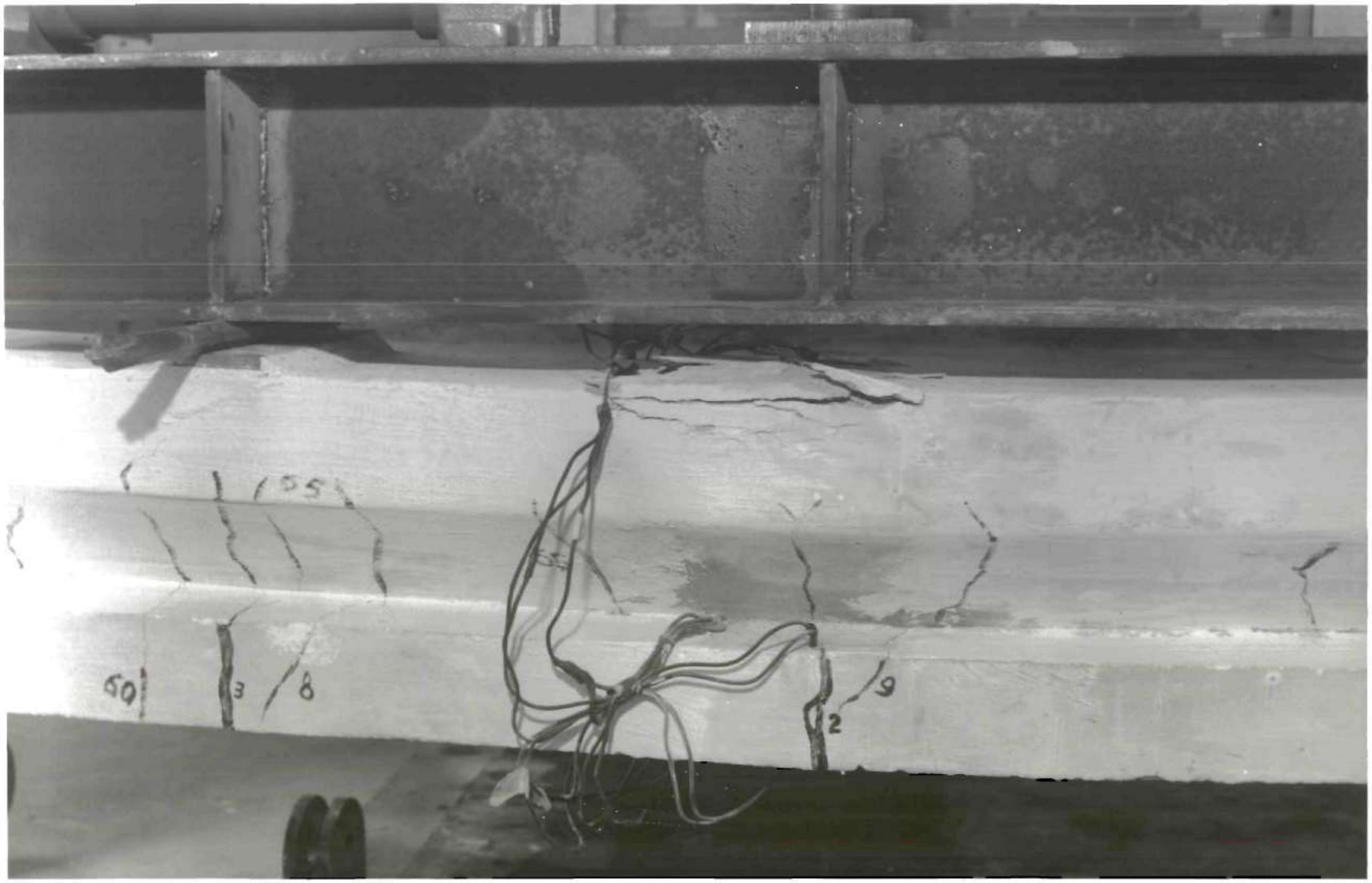


Figure 31. Ultimate Failure of Control Beam

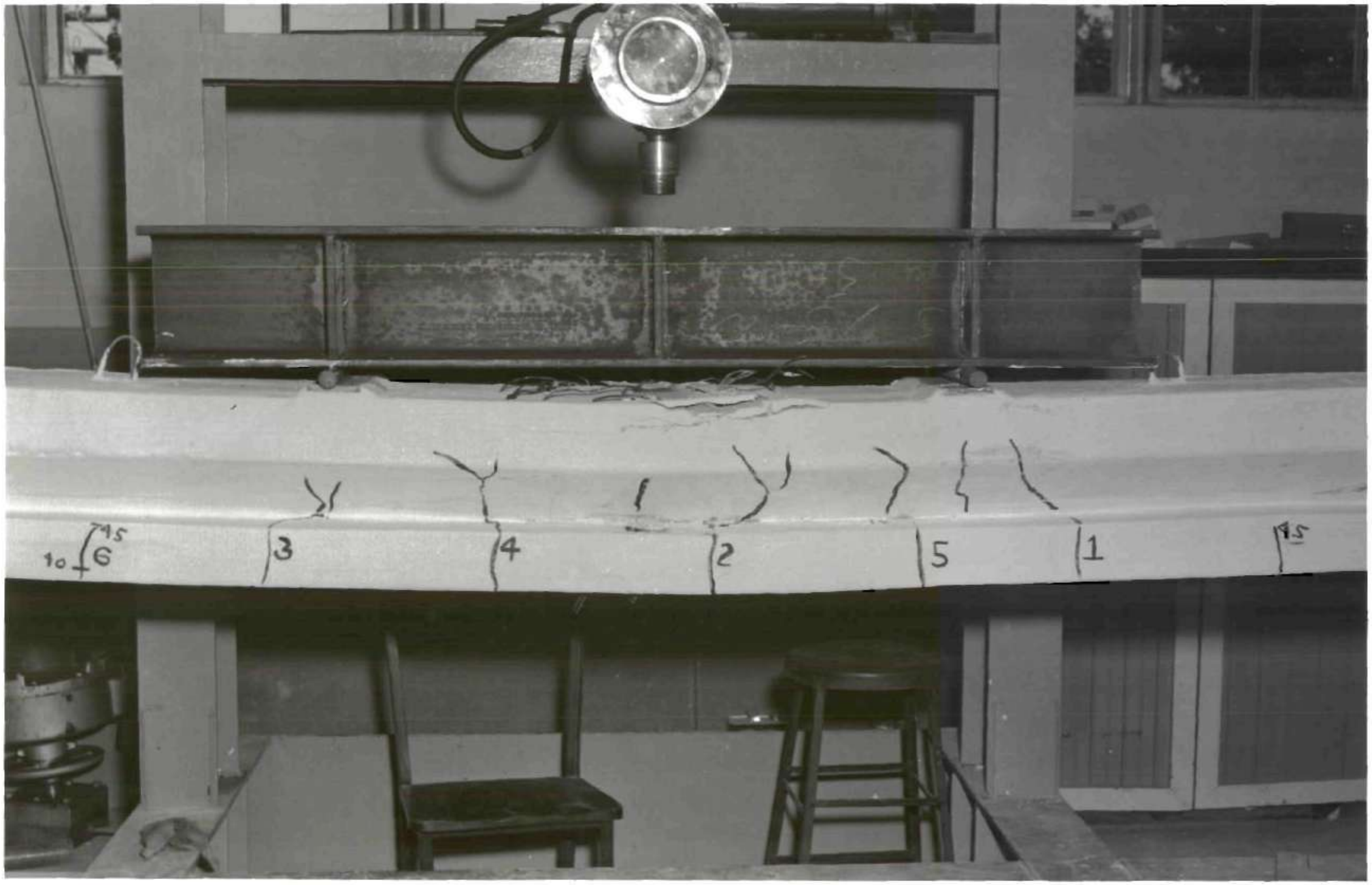


Figure 32. Ultimate Failure of Preflexed Beam #2

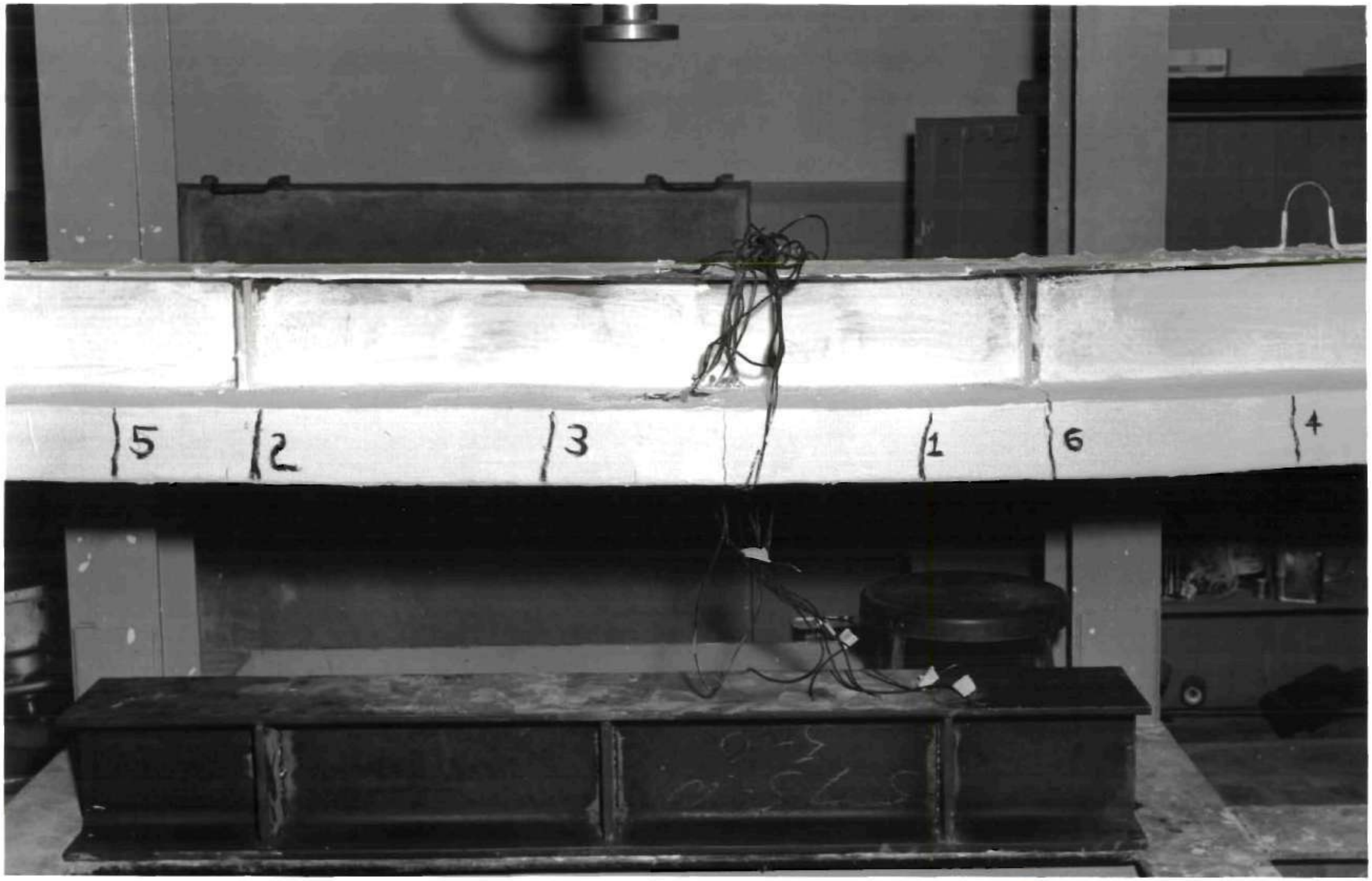


Figure 33. Ultimate Failure of Preflexed Beam

defined for any of the beams tested. No well defined yield point was observed on any of the plots of strain versus load, or deflection versus load. The only well defined changes in the proportionality of strain with load occurred in the strains in the lower flanges when the lower flange concrete cracked. However, the variation of strain with load in the steel continued to be linear, above the cracking load, as it was below it, as can be seen in Figures 25 and 26. This, of course, was not true for the lower flange concrete. After it cracked, the strain was roughly constant regardless of the increase in load.

For beams #1 and #2, the limit of proportionality of strain to load was roughly 40,000 pounds. At loads greater than 40,000 pounds, the strain gradually began to increase faster than the load. When a load of 50,000 pounds had been reached, strains and deflections were increasing very rapidly with only small increases in load.

For beam #3, strain in the uncovered upper steel flange reached its proportional limit, corresponding to a measured stress of only 36,900 p.s.i., was caused by a lack of lateral rigidity of the upper (compression) flange.

In the lower (tension) steel flange of beam #3, the proportional limit was in excess of 40,000 pounds load, or about the same as in beams #1 and #2. The strain in the lower flange concrete was again constant at loads greater than the cracking load of 25,000 pounds.

CHAPTER VI

CONCLUSIONS

The conclusions reached as a result of the tests described in this paper pertain to two phases of the preflex method. The first phase deals with the elastic theory, and its use as a method of analyzing the stresses which will occur in preflexed beams. The second phase deals with the advantages of using preflexed beams instead of plain steel joists encased in concrete.

Elastic Theory

The tests indicate that the elastic theory as it applies to beams of two materials of different elastic properties may be used to predict stresses which will occur in a preflexed beam when the preflexing load is applied and when it is removed. It may also be used to predict the changes in stresses which will occur in preflexed beams when they are subjected to working loads. Conversely, the tests indicate that the elastic theory cannot be used to predict the amount of prestress loss which will occur, nor the initial stress condition in a preflexed beam before working loads are applied.

Stresses caused by preflexing and unloading.--Measured stresses agreed with computed stresses fairly well during the preflexing and unloading operations. When the preflexing load was applied to the plain steel beam, the maximum variation between measured stresses and computed stresses was nine percent for both beams. Immediately after removing the preflexing load from the steel and concrete beam, the maximum variation between measured and computed stresses was 21 percent in the steel and 32 percent in the concrete, when stresses were of appreciable magnitude. These variations are sufficiently small to make it safe to design preflexed beams by the elastic theory if the usual allowable stresses in steel and concrete are used.

Prestress loss.--Prestress loss as shown by strain measurements was not consistent between beam #2 and beam #3. It is believed that the strain gage readings on the upper flange of beam #3 were in error, since a stress increase of 9,400 p.s.i. in four days was impossible. Strain gages on the lower steel flanges indicated a steel pretension loss of between 39 percent and 60 percent in four days. Strain gages on the bottom of the lower flange concrete indicated a concrete plastic flow of between 53 percent and 105 percent of the original elastic strain in four days. Considerable prestress loss in the four day period is indicated, even if no quantitative reliance is placed upon the data.

Because of the poor quality of the prestress loss data, no recommendations are made concerning the amount of prestress loss which may be expected when using the preflex method. It should be mentioned, however, that prestress loss was extremely high on the beams tested.

Stresses caused by applied loads.--Measured stresses caused by changes in load on beams #2 and #3 agreed fairly well with changes in stresses computed by the elastic theory. In general, agreement was better for beam #2 than for beam #3. For beam #2, the maximum variation between measured and computed stresses in steel was 28 percent and in concrete 31 percent. For beam #3, the maximum variation between measured and computed stresses, was, in steel, 44 percent and in concrete, 12 percent. The average variations were much smaller than the maximum variations.

Agreement between stresses measured and stresses computed is good enough to allow preflexed beams to be designed for their working loads by the elastic theory. If conventional allowable stresses are used in the design of preflexed beams, the safety factors inherent in those allowable stresses will offset the unsafe variation between actual stresses and design stresses.

Advantages of Preflexed Beams

Results of the tests described herein indicate that preflexed beams, when loaded with working loads, have several advantages over plain steel joists encased in concrete. Preflexed beams, when compared with non-preflexed beams, exhibit less cracking, less deflection, and less strain. When loaded to the ultimate, however, preflexed beams exhibit no advantages over non-preflexed beams.

Resistance to cracking.--As indicated by the loads at which cracking of the lower flange concrete occurred, the preflexed beams supported over twice the load, without cracking, that the non-preflexed beam supported. Also, when loads were removed in the course of load tests on the beams, cracks in the preflexed beams closed completely, while cracks in the non-preflexed beam did not. This fact indicated that an occasional overload would not permanently crack the preflexed beam as it would the plain beam. It may be concluded that the cracking load was more than doubled by preflexing.

Beam stiffness.--At working loads, preflexing increased beam stiffness.

When loaded with the working load, beam #2 experienced 17 percent less center deflection than the non-preflexed control beam. With no upper flange concrete in place, preflexed beam #3 experienced 17 percent less deflection than a plain 8 WF 17 steel beam.

At loads less than the initial cracking load of 20,000 pounds, the preflexed beams evidenced an even greater increase in stiffness. At 20,000 pounds, beam #2 experienced 25 percent less deflection than beam #1, and beam #3 experienced 25 percent less deflection than a plain 8 WF 17 steel beam.

Preflexed beams are, therefore, most efficient regarding deflection if they are loaded so their lower flange concrete remains uncracked. Deflection for a given load was reduced 25 percent when the beam remained uncracked, and was reduced only 17 percent after the beam had cracked. Loads vs. deflections for all beams are plotted and compared in Figure 24.

Beam strain.--For any applied load, strain in the preflexed beam #2 was smaller than similar strain in the control beam. The reduction in strain was greatest in the lower steel flange, and least in the upper steel flange. Again, the reduction in strain was greater for loads smaller than the cracking load, than for loads larger than the cracking load.

At a load of 20,000 pounds, beam #2 experienced 59 percent less strain in the lower steel flange than beam #1. At the working load of 28,900 pounds, the difference was only 40 percent.

Cracking of the lower flange concrete did not visibly affect the strain in the upper steel flange. For any applied load to 40,000 pounds, beam #2 had between six percent and seven percent less strain in the upper steel flange than beam #1.

Concrete strain in the upper flange was reduced a maximum of 38 percent by preflexing. Concrete strain in the lower flange became constant after the lower flange concrete cracked, and no comparison of strain was made.

When beam #3 was compared with beam #2, the only significant difference in strain was in the upper steel flange. Since beam #3 had no concrete around this flange, the strain was much greater. At working loads the upper steel strain in beam #3 was 330 percent of that in beam #2. Beam #3 eventually failed by lateral buckling of the upper steel flange without developing the ultimate compressive strength of the flange. That failure indicates that concrete should be placed around the compression flange to prevent buckling, even if it is not needed to resist direct compression.

It was proven by the strain measurements that preflexing increases beam stiffness by reducing the strain on the lower steel flange of the beam. It was also proven that concrete must be placed around the compression flange to prevent buckling, if the full potential strength of a preflexed beam is to be developed.

Ultimate load.--Preflexing did not increase the ultimate strength of any beam tested. There was a difference of only 100 pounds between the ultimate loads of beam #1 and beam #2. After the prestressed lower flange concrete cracked, the prestressed beams behaved exactly like the nonprestressed beam. After the applied load became larger than the cracking load, all advantages of prestressing were lost, and the two beams behaved alike. Tension cracks became longer as the load increased, until the compression concrete, reduced in area, failed. Tension cracks ran horizontally along the joint between upper and lower concrete. These cracks indicated relative movement between the two masses of concrete, and hence a bond failure between steel and concrete.

In beam #3, ultimate failure occurred by lateral buckling of the upper steel flange. After the prestressed lower concrete cracked, this beam acted as a plain steel beam. Lateral bracing of the compression flange would have increased the ultimate load considerably.

These tests proved that preflexing does not increase the ultimate strength of a beam, but it does increase allowable working loads by reducing deflection, strain, and undesirable cracking.

CHAPTER VII

RECOMMENDATIONS

As a result of the tests conducted on preflexed beams, several conclusions were reached. Based on these conclusions, recommendations are made concerning a tentative procedure for the design of preflexed beams, and concerning additional research which is felt to be necessary.

Design Method

The tests indicated that preflexed beams may be designed using the elastic theory in combination with usual allowable working stresses. It is recommended that all preflexed beams designed by the elastic theory be made to conform to the specifications contained in either the 1951 ACI Building Code, or the 1940 Joint Code, insofar as is possible.

Materials.-- For greatest efficiency, high strength steel and quality concrete should be used in the construction of preflexed beams. The steel should be similar to structural nickel steel (A.S.T.M.-A8-46), or Mayari R manufactured by Bethlehem Steel Co. (See Appendix A, Table 3.) Concrete which is to be prestressed should have a 28 day cylinder strength of 6000 - 7000 p.s.i. Concrete which is to be placed around the compression flange of the steel and will not be prestressed should have a 28 day cylinder strength of 2500 - 3000 p.s.i., since its most important function is to brace the steel compression flange.

The concrete which is to be used in the construction of the preflexed beams should be tested before construction is begun, to make sure that it has the strength and elastic properties specified by the designer. This is especially important in the case of the modulus of elasticity of the high strength concrete which is to be prestressed. The normal expression for the modulus of elasticity, E_c, as given by the expression:

$$E_c = 1000\ f'_c \tag{8}$$

does not apply to high strength concrete. An expression developed by Sutherlund and Reese for E_c:

$$E_c = 1{,}800{,}000 \quad 460\ f'_c \tag{9}$$

gives better results for high strength concrete, but is still not too reliable. (12) If at all possible, E_c should be determined by stress strain tests made on the concrete to be used in the preflexed beams.

Allowable stresses.--Allowable concrete stresses, both during preflexing and during the service life of preflexed beams should be limited to 45 percent of the 28 day cylinder strength, as required by the ACI and Joint Codes. Stresses higher than this will result in excessive plastic flow and loss of prestress, and possibly in an unsafe structure.

Allowable steel stresses may be increased during preflexing to 75 percent of yield strength, if preflexing loads are applied by jacks. The allowable stress here is limited by lateral deflection of the unbraced compression flange, and not by beam failure. These high stresses are permitted at preflexing, because the preflexing loads can be accurately predicted and closely controlled. If these loads are applied by jacking, as recommended, beam failure would result only in the inability of the beam to resist the full preflexing load. A dangerous condition would not result. If the compression flange could be adequately braced, there is no reason why the beam could not be stressed to 100 percent of its yield strength during preflexing. Also, if properly designed, the preflexing load is the greatest load which will ever be applied to the beam, and it is applied when the beam has its smallest moment resisting capacity.

Allowable steel stresses caused by working loads should be limited to 55 percent to 60 percent of the yield strength, or 30,000 p.s.i. for nickel steel and Mayari R.

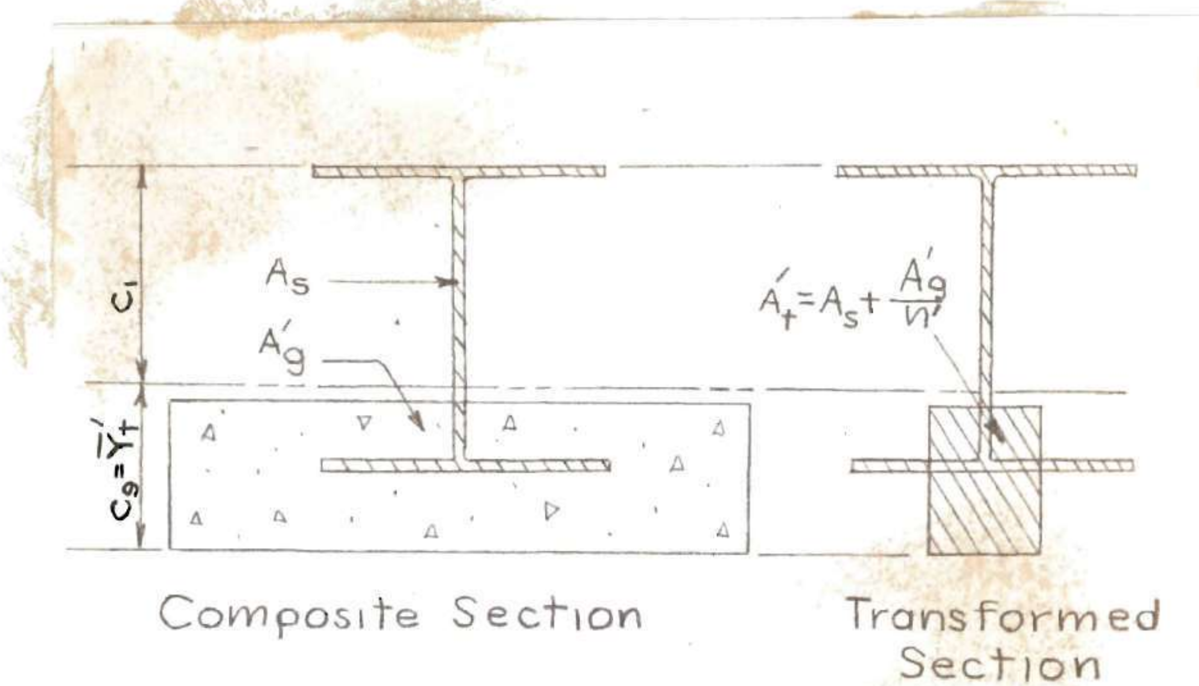
Computation of stresses.--The design of a preflexed beam by the elastic theory is done by trial and error. With the span and working loads known, a steel beam is chosen which will carry those loads with maximum stress of 75 percent of the yield stress. The working loads become the preflexing loads, and are applied to the steel beam so that the working load moment diagram is duplicated. Steel stresses, f_s, caused by the preflexing load are computed for the beam chosen, by

$$f_s = \frac{Mc}{I} \tag{10}$$

A trial area of concrete is then placed around the tension flange of the preflexed steel beam. Knowing the moduli of elasticity of concrete, E_c', and of steel E_s, the modular ratio, n', is computed by the expression

$$n' = \frac{E_s}{E_c'} \tag{7}$$

Knowing the modular ratio, n, the trial cross section of steel and concrete is transformed to an equivalent cross section of steel by dividing the gross concrete area A'_g by n', as shown in Figure 34.



Computation of Equivalent Steel Section

Figure 34

Location of the centroidal axis, $\bar{y}'_t$, and the moment of inertia, I, are computed for the transformed section. Knowing these properties, the changes in stress caused by removing the preflexing load are computed in the steel by the expression

$$f'_s = \frac{M'c'}{I'_t} \tag{11}$$

and in the concrete by the expression

$$f'_c = \frac{M'c'}{I'_t\, n'} \tag{12}$$

If the maximum compressive stresses in the concrete exceed $0.45\ f'_c$, a larger concrete area must be assumed, and the process repeated. This is continued until a concrete area is obtained which will not be overstressed by the removal of the preflexing load. Then, by adding the change in stress caused by removing the preload, f'_s, to the stress originally caused by applying the preload f_s to the steel, (zero in concrete) we obtain the stress in both steel and concrete after the preload is removed. Mathematically we have

$$f_{s_u} = \frac{Mc}{I} + \frac{M'c'}{I'_t} \tag{13}$$

and

$$f_{c_u} = \frac{M'c'}{I'_t n'} \tag{14}$$

where the subscript "s" refers to steel, and "c" refers to concrete. Tension is considered positive and compression negative.

The next step is to design the upper flange concrete. This will normally be poorer quality than the lower flange concrete, and therefore have a higher modular ration n". The procedure is the same as for designing the lower flange concrete. A trial section is assumed, an equivalent steel section obtained, and the location of the neutral axis, $\overline{Y}''_t$ and the moment of inertia I''_t computed. Then, the changes in stress, f", caused by applying the working load are computed for the steel by the expression

$$f''_s = \frac{M''c''}{I''_t} \tag{15}$$

and for the concrete by

$$f''_c = \frac{M''c''}{I''_t n''} \tag{16}$$

If the maximum changes in stress in the upper flange concrete exceed $0.45\ f'_c$, a larger concrete area must be assumed and the process repeated until acceptable stresses are obtained. The final stresses obtained under working load are expressed by:

$$f_{s_w} = \frac{Mc}{I} + \frac{M'c'}{I'_t} + \frac{M''c''}{I''_t} \qquad \text{for steel} \qquad (17)$$

$$f'_{c_w} = 0\ \frac{M'c'}{I'n'} + \frac{M''c''}{I''n'} \qquad \text{for lower flange concrete} \qquad (18)$$

$$f''_{c_w} = 0 + 0 + \frac{M''c''}{I''n''} \qquad \text{for upper flange concrete} \qquad (19)$$

So far, the design procedure has neglected the effects and magnitude of prestress loss which takes place at a gradually diminishing rate after the removal of the preflexing load. Steel strain readings indicated a loss of 40 percent to 60 percent of pretension in the lower flange in four days. Measured concrete strain at cracking loads indicated a loss of 30 percent of the initial prestress. On the basis of the limited tests made, a prestress loss of at least 40 percent should be allowed for in the design of preflexed beams. The value of 40 percent is extremely tentative, and should not be relied upon without conducting further research. On the basis of present knowledge, however, it is the value which should be recommended.

This means that only 60 percent of the initial concrete prestress will be available to resist tension in the lower flange. Revising equations 17, 18, and 19, we have

$$f_{s_w} = 0.6\left[\frac{Mc}{I} + \frac{M'c'}{I_t'}\right] + \frac{M''c''}{I_t''} \tag{20}$$

$$f'_{c_w} = 0.6\,\frac{M'c'}{I_t'n'} + \frac{M''c''}{I_t''n'} \tag{21}$$

$$f''_{c_w} = \frac{M''c''}{I''} \tag{22}$$

Need for Additional Research

There is a great need for additional research into the problem of prestress loss in preflexed beams. The tests described in this thesis produced little reliable information about it. It is felt that the 30 to 60 percent loss indicated by the tests is excessive. Prestress loss normally is about 10 percent or 13 percent in conventional prestressed concrete. Long time strain and deflection readings on unloaded preflexed beams would probably indicate the true amount of prestress loss which occurs in preflexed beams. It is recommended that bond and shrinkage conditions be investigated over a long period of time to determine their effects on prestress loss.

In any future preflexed beams constructed for test purposes, it is suggested that full sized beams be employed on spans of thirty or forty feet. Also larger sized aggregate should be used than the 3/8 maximum size employed in these tests. The use of high early strength concrete speeds the construction of preflexed beams, but the concrete should be carefully designed and well cured.

A P P E N D I X - A

Preflexed Beam Test Data

Table 3 Physical and Chemical Properties of Steel Used in Construction of Preflexed Beams

Heat No.	44E231
Description	B8 x 17
Thickness	0.206"
Yield Point	56,960 p.s.i.
Tensile Strength	94,610 p.s.i.
Elongation	16.8%
Chemical Analysis	(%)
C	0.12
Mn	0.89
P	0.095
S	0.026
Si	0.32
Ni	0.44
Cr	0.58
Cu	0.45
Bends	OK

Table 4. Measured Stresses and Strains Caused by Preflexing Beam Number 2

Load (lbs)	Gages 1 & 2 Strain (Micro-in/in)	Gages 1 & 2 Stress (p.s.i.)	Gages 3 & 4 Strain (Micro-in/in)	Gages 3 & 4 Stress (p.s.i.)	Gages 5 & 6 Strain (Micro-in/in)	Gages 5 & 6 Stress (p.s.i.)	Remarks
0	0	0	0	0	0	0	
28,900	-1320	-39,600	1330	39,900	860	25,800	Preflexing Load
28,900	-1344	-40,300	1382	41,400	879	26,400	Preflexing load applied for a week
0	- 55	- 1,600	695	20,800	560	16,800	Preflexing load removed
0	- 42	- 1,300	471	14,100	418	12,500	2 day prestress loss
0	- 36	- 1,100	423	12,700	392	11,800	4 day prestress loss
0	- 97	- 2,900	424	12,700	319	9,600	6 day loss-top flange concrete applied
0	- 8	- 200	494	14,800	400	12,000	9 day loss-top concrete applied

Table 4. Continued

Load	Gages 7 & 8		Gages 9 & 10		Gages 11 & 12		Remarks
(lbs)	Strain (Micro-in/in)	Stress (p.s.i.)	Strain (Micro-in/in)	Stress (p.s.i.)	Strain (Micro-in/in)	Stress (p.s.i.)	
0	0	0	0	0	0	0	
28,900	540	16,200					Preflexing load
28,900	553	16,600	0	0	0	0	Preflexing load applied for one week
0	468	14,000	-133	-366	-977	-2690	Preflexing load removed
0	354	10,600	-108	-300	-1267		2 day prestress loss
0	312	9,300	- 77	-210	-1500		4 day prestress loss
0	277	8,300					6 day loss-top concrete applied
0	330	9,900					9 day loss-top concrete applied

Table 5. Measured Stresses and Strains Caused by Preflexing Beam Number 3

Load (lbs)	Gages 1 & 2 Strain (Micro-in/in)	Gages 1 & 2 Stress (p.s.i.)	Gages 3 & 4 Strain (Micro-in/in)	Gages 3 & 4 Stress (p.s.i.)	Gages 5 & 6 Strain (Micro-in/in)	Gages 5 & 6 Stress (p.s.i.)	Remarks
0	0	0	0	0	0	0	
28,900	-1415	-42,400	1350	40,500	934	28,000	Preflexing load
28,900	-1350	-40,500	1263	37,800	791	23,700	Preflexing load applied for one week
0	- 123	- 3,700	708	21,300	556	16,700	Preflexing load removed
0	- 241	- 7,200	382	11,500	239	7,200	2 day prestress loss
0	- 332	-10,000	287	8,600	196	5,900	6 day prestress loss

Table No. 5 Continued

Load	Gages 7 & 8		Gages 9 & 10		Gages 11 & 12		Remarks
(lbs)	Strain (Micro-in/in)	Stress (p.s.i.)	Strain (Micro-in/in)	Stress (p.s.i.)	Strain (Micro-in/in)	Stress (p.s.i.)	
0	0	0	0	0	0	0	
28,900	586	17,600					Preflexing load
28,900	543	16,300	0	0	0	0	Preflexing load applied for one week
0	484	14,500	-113	-310	-1073	-2950	Preflexing load removed
0	191	5,730	-395	-1085	-1437		2 day prestress loss
0	115	3,400	-450	-1240	-2200		6 day prestress loss

Table 6. Strains in Beam #1 Caused by Load Test

Load in pounds	Measured Strain in Micro-inches/inch Gage # 1 & 2	3 & 4	5 & 6	7 & 8	13 & 14
0	0	0	0	0	0
10,100	- 155	339	241	183	- 220
19,900	- 280	980	690	520	- 580
30,000	- 489	1209	885	690	- 917
40,000	- 664	1568	1165	865	-1155
50,000	-1104	1860	5605	3305	-2022
60,000	-1300		6345	4010	-2228
0	- 370		4540	2770	- 894
9,860	- 528		4860	3000	-1116
14,700	- 631		5004	3111	-1277
30,100	- 877		5462	3437	-1617
40,100	-1014		5765	3652	-1868
49,800	-1194		6100	3887	-2094
62,200	-1371		6535	4196	-2406
0	- 408		4740	3390	- 910
10,400	- 586		5036	3604	-1189
20,400	- 759		5337	3823	-1442
30,300	- 933		5640	4030	-1681
40,400	-1099		5946	4250	-1939
50,700	-1254		6290	4493	-2230
60,500	-1417		6710	4805	-2402
66,800			Ultimate Load		

Table 7. Strains in Beam #2 Caused by Load Test

Load (lbs)	Measured Strain in Micro-inches/inch Gage #						
	1 & 2	3 & 4	5 & 6	7 & 8	9 & 10	11 & 12	13 & 14
0	0	0	0	0	0	0	0
5,000	- 60	65	40	30	40	130	- 70
10,000	- 140	135	90	60	75	190	- 160
15,000	- 215	220	150	90	125	340	- 225
20,000	- 295	315	210	140	180	590	- 360
25,000	- 380	535	370	275	220	630	- 495
0	- 25	40	40	30	75	225	15
25,000	- 385	545	380	280	220	605	- 485
30,000	- 465	715	500	370	220	660	- 605
35,000	- 550	945	670	500	205	660	- 755
40,000	- 645	1130	810	610	205	660	- 860
45,000	- 735	1375	990	755	205	660	-1010
50,000	- 835	1685	1250	960	195	660	-1175
0	- 55	240	120	105	140		- 15
50,000	- 860	1750	1280	995	145		-1205
55,000	- 965	1980	1580	1395	195		-1515
60,000	-1170	2200	1970	2330	360		-2145
66,700			Ultimate Load				

Table 8. Strains in Beam #3 Caused by Load Test

Load (lbs)	Measured Strain in Micro-inches/inch Gage # 1 & 2	3 & 4	5 & 6	7 & 8	9 & 10	11 & 12
0	0	0	0	0	0	0
5,000	- 205	75	25	- 5	15	120
10,000	- 400	150	55	0	20	270
15,000	- 605	235	90	0	25	375
20,000	- 805	320	135	0	40	515
0	- 10	5	5	5	20	0
20,000	- 815	325	145	10	55	530
25,000	-1035	520	265	90	30	605
30,000	-1285	780	445	200	40	525
35,000	-1510	1000	600	295	35	515
40,000	-2095	1230	750	380	5	515
0	- 385	- 40	- 15	- 25	5	- 15
40,000	-2130	1255	780	400	10	415
45,000	-2690	1520	940	490	0	435
48,800			Ultimate Load			

Table 9. Beam Deflections Caused by Load Tests

Beam #1			Beam #2			Beam #3		
Load (lbs)	Deflection in inches ℄	Third points	Load (lbs)	Deflection in inches ℄	Third points	Load (lbs)	Deflection in inches ℄	Third points
0	0.000	0.000	0	0.000	0.000	0	0.000	0.000
10,000	0.106	0.095	5,000	0.057	0.055	5,000	0.113	0.106
19,900	0.254	0.229	10,000	0.097	0.091	10,000	0.180	0.168
30,000	0.376	0.331	15,000	0.138	0.128	15,000	0.245	0.228
40,000	0.499	0.448	20,000	0.179	0.167	20,000	0.312	0.289
50,000	0.922	0.820	25,000	0.244	0.224			
60,000								
0	0.400	0.350	0	0.017	0.015	0	0.005	0.000
9,860	0.538	0.475	25,000	0.248	0.227	20,000	0.315	0.290
14,700	0.698	0.628	30,000	0.300	0.274	25,000	0.400	0.368
30,100	0.862	0.671	35,000	0.362	0.331	30,000	0.489	0.446
40,100	0.936	0.805	40,000	0.427	0.388	35,000	0.582	0.530

Table 9. Continued

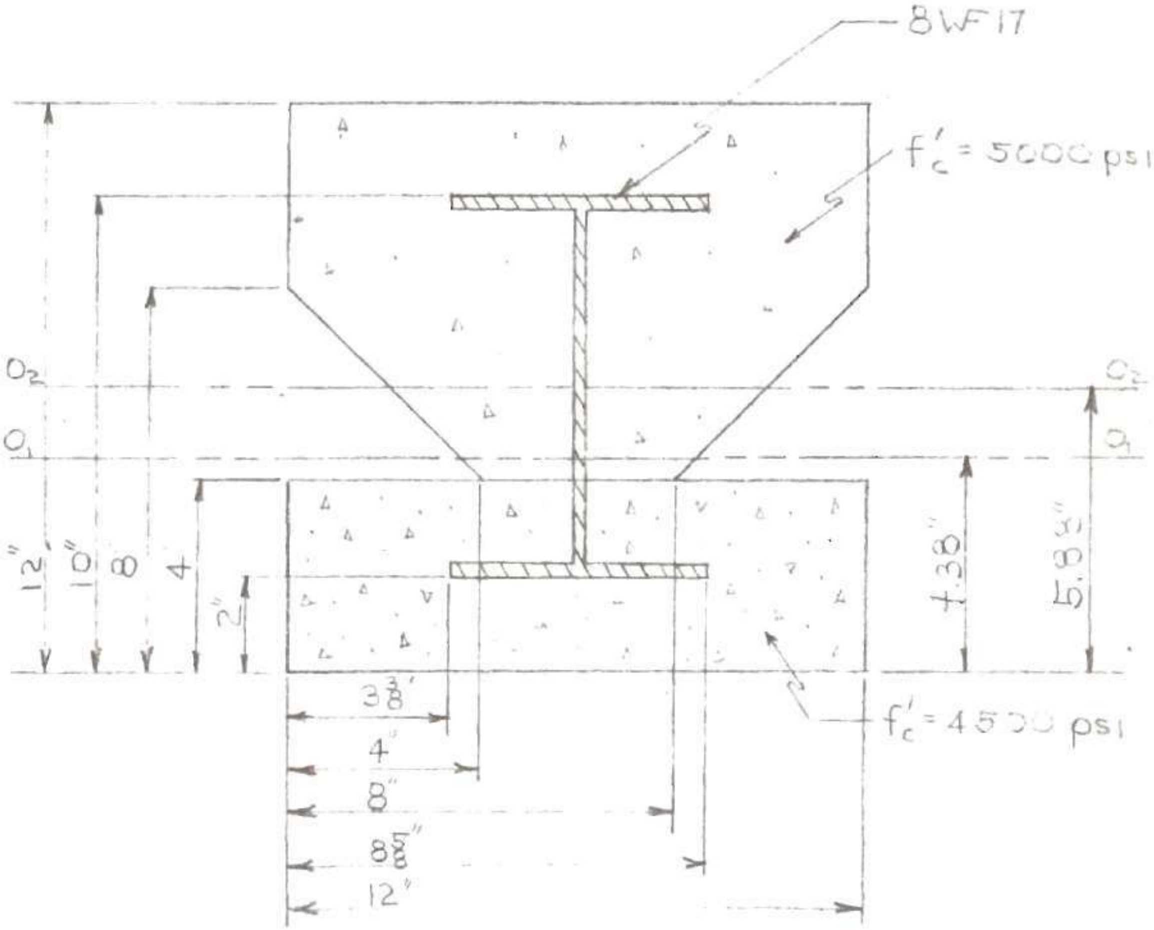
Beam #1			Beam #2			Beam #3		
Load (lbs)	Deflection in inches ℄	Third points	Load (lbs)	Deflection in inches ℄	Third points	Load (lbs)	Deflection in inches ℄	Third points
49,800			45,000	0.496	0.452	40,000	0.708	0.642
62,200			50,000	0.581	0.527			
66,800	$2\frac{3}{16}$							
			0	0.065	0.062	0	0.048	0.059
			50,000	0.597	0.544	40,000	0.716	0.650
			55,000	0.718	0.647	45,000	0.862	0.793
			60,000	0.995	0.888	48,800		
			66,700	$2\frac{1}{4}$				

Table 10. Changes in Stresses Computed from Measured Strains

Load (lbs)	Computed Stresses in p.s.i. at Gages: 1 & 2	3 & 4	5 & 6	7 & 8	9 & 10	11 & 12	13 & 14
				Beam #1			
0	0	0	0	0	0	0	0
20,000	- 8,400	29,400	20,700	15,600			-1330
30,000	-14,700	36,300	26,600	20,700			-2110
40,000	-19,900	47,000	35,000	25,900			-2680
				Beam #2			
0	0	0	0	0	0	0	0
20,000	- 8,800	9,400	6,300	4,200	500	1640	- 830
25,000	-11,100	16,100	9,100	8,200	610	1750	-1140
30,000	-14,000	21,500	15,000	11,100	610	1840	-1390
40,000	-19,400	33,900	24,300	18,300	575	1840	-1980
				Beam #3			
0	0	0	0	0	0	0	0
20,000	24,300	9,600	4,100	300	110	1430	
25,000	30,100	15,600	7,900	2,700	83	1680	
30,000	38,700	23,400	13,300	6,000	110	1460	
40,000	63,000	36,900	22,500	11,400	0	1430	

APPENDIX 'B' COMPUTATION OF THEORETICAL BEAM PROPERTIES

BEAM DIMENSIONS



EXPERIMENTAL MODULUS OF ELASTICITY, E_c

LOWER FLANGE

$$E_c = \frac{\Delta f_c}{\Delta \epsilon} = \frac{2860}{1035} = \underline{2.76 \times 10^6 \text{ p.s.i.}}$$

$$n' = \frac{E_s}{E_c} = \frac{30}{2.76} = \underline{10.9}$$

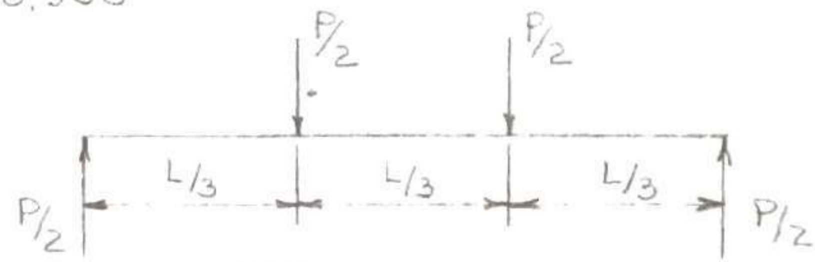
UPPER FLANGE

$$E_c = \frac{2860}{1255} = \underline{2.28 \times 10^6 \text{ p.s.i.}}$$

$$n'' = \frac{E_s}{E_c} = \frac{30}{2.28} = \underline{13.1}$$

PREFLEXING STRESSES

$P = 28{,}900^{\#}$



$$M = \frac{PL}{6} \quad ; \quad f = \frac{M}{S}$$

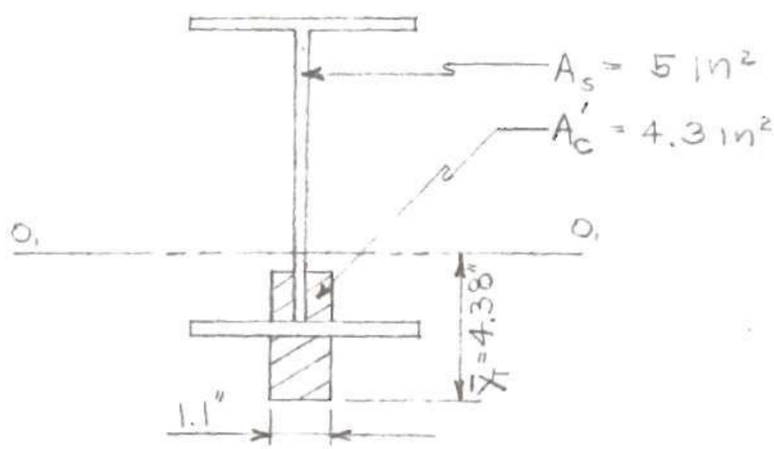
$$\text{STEEL STRESS, } f_s = \frac{PL}{6S} = \frac{28900 \times 9.5 \times 12}{6 \times 14.1}$$

$$f_s = \underline{39{,}000 \text{ p.s.i.}}$$

CONCRETE STRESS CAUSED BY UNLOADING

$$A_c' = 48 - 1 = 47 \text{ in}^2$$

$$A_t' = \frac{A_c'}{n'} = \frac{47}{10.9} = 4.3 \text{ in}^2$$



EQUIVALENT STEEL SECTION

$$\bar{Y}_t = \frac{\Sigma AY}{\Sigma A} = \frac{5 \times 4}{5 + 4.3} = 2.38 \text{ in}$$

TRANSFORMED SECTION MOMENT OF INERTIA, I_t'

$$I_o = 56.4 \text{ in}^4$$

$$A_s d^2 = 5 \times 1.62^2 = 13.1$$

$$\frac{bd^3}{12n} = \frac{12 \times 4^3}{12 \times 10.9} = 5.9$$

$$Ad^2 = 4.3 \times 2.38^2 = 24.5$$

$$I_t' = \underline{99.9 \text{ in}^4}$$

$$f'_{c_{9,10}} = \frac{\Delta P' L c'}{6 I_t' n'} = \frac{28{,}900 \times 9.5 \times 12 \times 0.38}{6 \times 99.9 \times 10.9} = \underline{190 \text{ psi at top of lower flange}}$$

$$f'_{c_{11,12}} \quad \frac{28{,}900 \times 9.5 \times 12 \times 4.38}{6 \times 99.9 \times 10.9} = \underline{2230 \text{ psi at bottom of lower flange}}$$

CHANGE IN STEEL STRESS CAUSED BY UNLOADING

$$\Delta f_s' = \frac{\Delta P' L c'}{6 I_t'} = \frac{28{,}900 \times 9.5 \times 12 \times 5.62}{6 \times 99.9} = \underline{30{,}500 \text{ psi in top flange of steel beam}}$$

$$\Delta f_s' = \frac{28{,}900 \times 9.5 \times 12 \times 2.38}{6 \times 99.9} = \underline{13{,}000 \text{ psi in lower flange of steel beam}}$$

<u>SUMMARY OF STRESSES CAUSED BY PREFLEXING</u>

$$f_{s_{U1,2}} = -39{,}000 + 30{,}500 = \underline{-8{,}500 \text{ psi}}$$

$$f_{s_{U2,3}} = 39{,}000 - 13{,}000 = \underline{26{,}000 \text{ psi}}$$

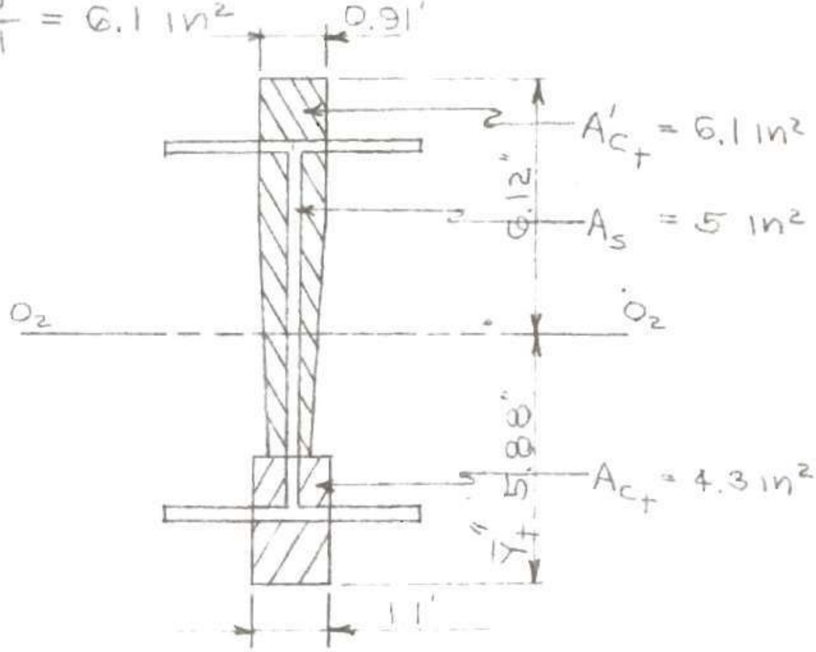
$$f_{c_{U9,10}} = \underline{-190 \text{ psi}}$$

$$f_{c_{U11,12}} = \underline{-2230 \text{ psi}}$$

SECTIONAL PROPERTIES OF THE COMPLETE BEAM

$A''_c = 8 \times 12 - 4 \times 4 = 80 \text{ in}^2$

$A''_t = \frac{80}{13.1} = 6.1 \text{ in}^2$



EQUIVALENT STEEL SECTION

$$\bar{Y}''_t = \frac{5 \times 6 + 4.3 \times 2 + 6.1\left[\frac{96 \times 8 - 16\left\{\frac{4}{3} + 4\right\}}{80}\right]}{5 + 4.3 + 6.1} = \underline{5.88''}$$

MOMENT OF INERTIA I''_t

$I_o \qquad = 99.9 \text{ in}^4$

$-bd^2 = -9.3 \times 1.5^2 \qquad = -20.9$

$\frac{bd^3}{12n'} = \frac{12 \times 8^3}{12 \times 13.1} \qquad = 39.1$

I'_o neglected

$bd^2 - 6.1 \times 2.12^2 \qquad = \underline{27.5}$

$I''_t \qquad = 145.6 \text{ in}^4$

CHANGES IN STRESSES CAUSED BY TEST LOADINGS OF BEAMS #1 & #2

$I_t'' = 145.6 \text{ in}^4$

$C''_{1,2} = 4.12''$; $C''_{3,4} = 3.88''$; $C''_{9,10} = 1.88$; $C''_{11,12} = 5.88$

$C''_{13,14} = 6.12''$

AT 20,000 LBS LOAD:

$$M = \frac{PL}{6} = \frac{20 \times 9.5 \times 12}{6} = \underline{380 \text{ inch-kips}}$$

$$\Delta f''_{s1,2} = \frac{380 \times 4.12}{145.6} = \underline{10,800 \text{ psi}}$$

$$\Delta f''_{s3,4} = 10.8 \times \frac{3.88}{4.12} = \underline{10,[illegible]00 \text{ psi}}$$

$$\Delta f''_{c9,10} = \frac{380 \times 1.88}{10.9 \times 145.6} = \underline{452 \text{ psi}}$$

$$\Delta f''_{c11,12} = \frac{380 \times 5.88}{10.9 \times 145.6} = \underline{1,410 \text{ psi}}$$

$$\Delta f''_{c13,14} = \frac{380 \times 6.12}{13.1 \times 145.6} = \underline{1,210 \text{ psi}}$$

AT 25,000 LBS LOAD

STRESS IS PROPORTIONAL TO LOAD

$$\Delta f''_{s1,2} = \frac{25}{20} \times 10800 = \underline{13,500 \text{ psi}}$$

$$\Delta f''_{s3,4} = \underline{12,[illegible]00 \text{ psi}}$$

$$\Delta f''_{c9,10} = \underline{560 \text{ psi}}$$

$$\Delta f''_{c11,12} = \underline{1,760 \text{ psi}}$$

$$\Delta f'_{c13,14} = \underline{1,510 \text{ psi}}$$

CHANGES IN STRESSES CAUSED BY TEST LOADINGS OF BEAM #3

$I_t = 99.9 \text{ in}^4$

$C_{1,2} = 5.62''$; $C_{3,4} = 2.38''$; $C_{9,10} = 0.38''$; $C_{11,12} = 4.38''$

BENDING MOMENT, M_1 = 380 Inch-kips FOR 20,000 LBS.

$$\Delta f''_{s1,2} = \frac{M}{I_1} \times C_{1,2} = \frac{380}{100} \times 5.62 = \underline{16\,900\ psi}$$

$$\Delta f''_{s3,4} = \frac{380}{100} \times 2.38 = \underline{9{,}050\ psi}$$

$$\Delta f''_{c9,10} = \frac{380}{100} \times \frac{0.38}{10.9} = \underline{132\ psi}$$

$$\Delta f''_{c11,12} = \frac{380}{100} \times \frac{4.38}{10.9} = \underline{1{,}530\ psi}$$

FOR THE 25,000 LB LOAD, STRESSES ARE PROPORTIONAL TO LOAD

$\Delta f''_{s1,2} = \underline{21{,}100\ psi}$ $\Delta f''_{c9,10} = \underline{165\ psi}$

$\Delta f_{s3,4} = \underline{11{,}300\ psi}$ $\Delta f''_{c11,12} = \underline{1{,}910\ psi}$

SUMMARY OF THEORETICAL STRESSES

STRESSES CAUSED BY:	STRESSES AT STRAIN GAGES (psi)				
	1&2	3&4	9&10	11&12	13&14
PREFLEXING BEAMS #2 & #3					
LOAD APPLICATION	-39,000	39,000	0	0	0
LOAD REMOVAL	-8,500	26,000	-190	-2230	
LOAD TESTS ON BEAMS #1 & #2					
0	0	0	0	0	0
20,000#	-10,800	10,100	452	1410	-1210
25,000#	-13,500	12,600	564	1760	-1510
LOAD TEST ON BEAM #3					
20,000#	-16,900	9,050	132	1530	—
25000#	-21,100	11,300	165	1910	—

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